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LIQUEFACTION MITIGATION TECHNOLOGY

ABSTRACT Seismically-induced soil liquefaction is a great threat to the Navy's huge shore facility investment located at the waterfront. Improvement of potentially liquefiable soil beneath and surrounding structures is currently feasible. Using empirical design methods, the construction of stone columns on relatively close centers provides increased resistance to lateral loading from earthquake waves and increases vertical drainage by reducing the radial flow path, which can increase the rate of dissipation of pore water pressure. A background investigation of state-of-the-art soil improvement methods was conducted. The Princeton Effective Stress Soil Model was evaluated as a tool to aid the design of soil improvement methods. Finite element models with and without stone columns were subjected to simulated earthquake excitation using DYNAFLOW, a finite element computer model developed at Princeton University to analyze dynamic soil-structure interaction problems. Results indicate that the stone columns improved site conditions and decreased damage due to vibratory motion. The results also indicated that each potentially liquefiable site is unique and will require individual analysis.

NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME CALIFORNIA 93043-5003

METRIC CONVERSION FACTORS

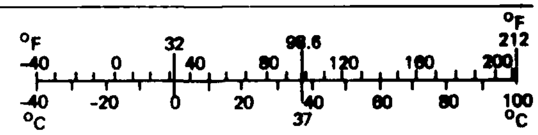
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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Seismically-induced soil liquefaction is a great threat to the Navy's huge shore facility investment located at the waterfront. Improvement of potentially liquefiable soil beneath and surrounding structures is currently feasible. Using empirical design methods, the construction of stone columns on relatively close centers provides increased resistance to lateral loading from earthquake waves and increases vertical drainage by reducing the radial flow path, which can increase the rate of dissipation of pore water pressure. A background investigation of state-of-the-art soil improvement methods was conducted. The Princeton Effective Stress Soil Model was evaluated as a tool to aid the design of soil improvement methods. Finite element models with and without stone columns were subjected to simulated earthquake excitation using DYNAFLOW, a finite element computer model developed at Princeton University to analyze dynamic soil-structure interaction problems. Results indicate that the stone columns improved site conditions and decreased damage due to vibratory motion. The results also indicated that each potentially liquefiable site is unique and will require individual analysis.

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INTRODUCTION

The Navy has huge facility investments in seismically active regions. Each year this investment is increased. The Navy, by the nature of its mission, must locate its facilities at the waterfront, encountering a high water table, and often on marginal foundation conditions.

The present study will consider state-of-the-art methods for mitigating seismically-induced soil liquefaction. Liquefaction plays a predominant role in waterfront damage, often being the single cause of widespread losses. Methods applicable for improving sites with existing structures will be evaluated for effectiveness using existing centrifuge test data. The usefulness of the finite element program DYNAFLOW to determine the feasibility of techniques for improving potentially liquefiable soil foundations beneath existing structures will also be considered.

BACKGROUND

If a saturated granular material is subjected to cyclic loading with the reversal of shear stresses, the material will tend to compact. If drainage is impeded, the tendency to decrease in volume will cause an increase in pore water pressure. If cyclic loading continues, the soil may reach a condition of zero effective stress and will suffer a partial or complete loss of strength which is termed liquefaction. Seismically-induced soil liquefaction is a great threat to the Navy's ability to carry out its mission in the event of a major earthquake.

Though the United States has not suffered a devastating earthquake in recent years, the seismic risk faced by the Navy is great, especially in the West and along the Pacific Rim. In Southern California, it is estimated that there is a 5 percent annual probability of a major event occurring that could affect a number of Naval bases.

The significance of liquefaction is evident in the following summaries when we assess the damage caused in recent major earthquakes.

1960 Chilean Earthquake (Magnitude 6-8.3)

The most spectacular damage occurred in Puerto Montt, to quay walls, steel sheet piles, and sea walls. Liquefaction of the loose fine sandy soils was the primary cause of the failures.

1964 Alaska Earthquake (Magnitude 8.4)

There was severe damage at Anchorage, Cordova, Homer, Kodiak, Seldovia, Seward, Valdez, Klawock, and Whittier. Large-scale landslides and liquefaction induced most of the extremely heavy damage and total destruction.

1964 Niigata Earthquake (Magnitude 7.5)

There was severe damage in Niigata Port (West Harbor). The areas affected were Additional Harbor, Yamanoshita Wharf, North Wharf, East Wharf, Central Wharf, South Wharf, Kurinoki River Landings, Bandai Island Wharf Shinano River Left Bank Bulkhead, and West Coast Bulkheads. Liquefaction caused most of the heavy damage.

1968 Tokachi-Oki Earthquake (Magnitude 7.8)

The ports affected were Hachinohe, Aomori, Hakodate, and Muroran. Damage was relatively light compared to that caused by the Niigata Earthquake. Most of the damage occurred to smaller scale structures. Liquefaction was not the primary cause of damage even though spouting sand sediments were seen at several waterfront areas near the damaged structures.

1973 Nemuro-Hanto-Oki Earthquake (Magnitude 7.4)

Severe damage occurred mainly in Hanasake and Kiritappu Ports. Nemuro Port, situated only 6 kilometers away from Hanasaki Port, sustained very slight damage. The damage was attributed to soil liquefaction.

1978 Mivagi-Ken-Oki Earthquake (Magnitude 7.4)

The areas affected were Shiogama, Sendai, and Ishinomake Ports, and Ishinomaki and Yuriage Fishing Ports. The damage in Ishinomaki Port accounted for approximately 90 percent of the total damage costs at port and harbor facilities caused by this earthquake. Gravity quay walls and piers suffered various degrees of damage. The damage to sheet pile quay walls was primarily due to liquefaction of fill materials. Liquefaction again played a significant role in this earthquake. At sites where liquefaction occurred, the damage to the port and harbor structures was severe. Conversely, the damage to port and harbor structures was small at sites where no liquefaction occurred.

Experience from recent earthquakes points out that liquefaction greatly increased the amount of damage incurred by waterfront facilities. To date, Naval facilities have generally escaped damage from earthquakes though the Navy did experience heavy damage at the Kodiak Naval Station during the 1964 Alaskan earthquake: 1 foot of differential settlement was noted beneath aircraft hangers; the sea wall received heavy damage; and the fill under asphalt aircraft ramps compacted. The damage was caused by soil failure due to the facility being constructed on 15 to 20 feet of fill where a seismically-induced pore pressure increase would be expected to reduce soil stiffness and shear strength. Although the United States has not had a large number of earthquakes exposing Navy facilities to damage, the Japanese have had several events demonstrating that seismic liquefaction is responsible for most waterfront damage.

Direct and indirect methods can mitigate liquefaction and assure safety of structures and personnel. With the exception of several dam sites, there has been essentially no experience with mitigation measures in liquefiable soils beneath existing structures. No general method is available which is applicable for all conditions. Each site is unique and will require a uniquely engineered solution.

SEISMICALLY-INDUCED SOIL LIQUEFACTION

During an earthquake, cohesionless soils that, under ordinary circumstances, provide adequate structural support may liquefy and settle. A loose sand subjected to seismically-induced vibratory motion tends to decrease in volume. If the sand is saturated and the drainage impeded, some of the interparticle stress is transferred to the water resulting in a rise in the pore water pressure. In general, the higher the intensity of the vibration, the greater the potential for an increase in pore water pressure. Shear resistance is lost as the pore water pressure approaches the confining pressure on the soil. Differential settlements and tilting may result, causing severe damage to structures. When the potential for soil liquefaction exists, the engineer has no means to evaluate the associated risk that could be caused by an earthquake.

Though early studies of liquefaction pertain to instabilities resulting from a gradual rise in the water table, the term "liquefaction" has been extended to include failure to soil under cyclic loading conditions induced by earthquake vibrations or blasts. Loss of soil strength may come about by static or dynamic loading, but shear stresses leading to liquefaction under cyclic loading may be much lower than shear stresses causing failure under static loading conditions. Continuous vibrations resulting in cyclic stresses cause a build-up of pore pressure which progressively reduces the effective strength. When the pore pressure is equal to the total stress (i.e., the effective stress equals zero), the sand has lost its shear strength and is considered liquefied. Liquefaction may then be defined in terms of a loss of strength and material transformation of a saturated granular material into a fluid.

The rate at which the pore pressure dissipates within a soil mass has a major influence on whether liquefaction will occur, particularly under cyclic loading. Pore pressure dissipation is a function of the radial drainage path, so the geometry of the soil mass is also important.

STATE-OF-THE-ART TECHNOLOGY IN LIQUEFACTION MITIGATION

There are three basic approaches to improve soil conditions to prevent liquefaction: (1) increase soil density, (2) increase effective confining pressure, and (3) use particulate or chemical grouting to increase the stiffness and fill voids, preventing a reorientation of soil particles to a denser state.

Table 1 lists mitigation measures and the site conditions where each could be used (Ref 1).

Table 1. Improvement Methods for Liquefiable Soil Conditions

Method	Case 1[a]	Case 2[b]	Case 3[c]
Blasting		X	X
Vibratory probe		X	X
Vibrocompaction grout	X	X	X[d]
Compaction piles	X	X	X
Heavy tamping (dynamic compaction)	X	X	
Displacement/compaction grout	X	X	X
Surcharge/buttress		X	X
Drains			
Gravel		X	X[d]
Sand		X	X[d]
Wick	X	X	X
Wells (permanent dewatering)	X	X	X
Particulate grouting	X	X	X
Chemical grouting	X	X	X
Pressure-injected lime	X	X	X
Electrokinetic injection	X	X	X
Jet grouting	X	X	X
Mix-in-place piles and walls	X	X	X
In-situ vitrification	X	X	X
Vibroreplacement stone & sand columns	X	X	X[d]
Root piles, soil nailing	X	X	X

[a] Beneath structures.
[b] Not-underwater free field adjacent to structure.
[c] Underwater free field adjacent to structure.
[d] The method has potential use for this case with special techniques required which would increase the cost.

The selection of potential methods for site improvement and the applications and results of the methods will depend on:

1. Location, area, depth, and volume of soil involved.
2. Soil type(s), properties, and conditions.
3. Site conditions.
4. Anticipated earthquake loading.
5. Structure type and condition.
6. Economic and social effects of the structure.
7. Availability of necessary materials (sand, gravel, admixtures, etc.).
8. Availability of equipment and skills.

The selection of potential methods also depends on the cost of the method or technique and the length of time needed to stabilize a site.

New Construction

Though to date no experience with remedial actions exists on potentially liquefiable soils beneath existing structures, great strides have been made in the area of ground improvement on new construction sites.

The basic concepts of soil improvement have been around for centuries and are valid today. These are drainage, drying, densification, cementation, reinforcement, and heating. The most significant developments in recent times are vibratory methods for densification of cohesionless soils, new soil reinforcement concepts, and new injection and grouting materials and procedures.

Extensive work has been done by Mitchell (Ref 2) in gathering data in the area of soil improvement to utilize marginal sites. Mitchell emphasizes the practical aspects of soil improvement, including consideration of soil types best suited for treatment, effective treatment depths, properties of treated soils, major applications, and relative costs. The following paragraphs briefly discuss the major areas of site improvement (e.g., compaction, consolidation, grouting, soil stabilization, thermal stabilization, and reinforced soil).

Compaction. Thick deposits of loose cohesionless soils run the risk of developing excessive total and differential settlements and, if saturated, liquefaction under dynamic loading. In many cases improvement can be achieved by densification. In-situ densification of loose, cohesionless layers is generally accomplished using dynamic compaction methods. This may be accompanied by displacement through the insertion of a probe and/or construction of a sand or gravel column. Methods include blasting, heavy tamping, and vibrocompaction (referring to compaction piles and to the insertion of vibratory probes into the ground).

Densification of cohesionless soil layers requires the initial soil structure be broken down and the particles rearranged in new packing arrangements. In saturated soils this may be accomplished by inducing liquefaction. Partially saturated soils are densified by collapse of the soil structure, allowing escape of gas from the voids.

Precompression. Weak and compressible soils, especially those that undergo a large decrease in volume and increase in strength under static load, are often strengthened by preloading prior to construction. Surcharge loads or vertical drains may accelerate the process.

Earth fills are most commonly used as the preload. Other preloads include water in tanks (or in lined ponds for larger areas), vacuum preload pumping from beneath an impervious membrane, anchor and jack systems, lowering of groundwater to increase consolidation pressure, and electro-osmosis. The latter methods are more complex than earth fills but reduce stability problems and the need for large volumes of surcharge fill.

Injection and Grouting. Grouting is most often used for groundwater control, ground strengthening, and ground movement control. Due to its high cost, grouting is generally limited to zones of relatively small volume and to special problems that cannot be solved by other methods.

The methods of injection are:

1. Permeation grouting in which the grout fills the soil pores.

2. Displacement grouting in which a stiff mixture fills the voids and compresses the surrounding soil.

3. Encapsulation in which fragmented ground (naturally or hydraulically) is injected by grout that coats but does not permeate the individual chunks of soil, forming a lens structure much like a card house.

Thermal Stabilization. Soil can be improved by heating or freezing. Moderate heating (100 °C) to fine-grained soils can cause drying and strengthening if rewetting is prevented. Heating to higher temperatures (600 to 1000 °C) can cause permanent improvement by decreasing water sensitivity, swelling, compressibility, and by increasing strength.

Frozen, wet soil is much stronger and less pervious than unfrozen ground. Temporary ground strengthening and support is accomplished by artificial ground freezing. One important application is freezing for support of open excavations. Some arctic areas require permanent ground freezing around pile foundations in permafrost and under heated buildings on permafrost.

Admixtures. The addition of admixtures to soil is the oldest method of ground improvement and its use is widespread today. Chemical admixtures, usually lime and cement, use ion exchange and cementation to improve the properties of soils.

Chemical additives improve volume stability (swelling and shrinking), strength and stress-strain properties, permeability, and durability. Volume stability is accomplished by replacing high hydration cations with low hydration cations, cementation, and waterproofing chemicals. Strength and stiffness are increased by eliminating large pores, bonding particles and aggregates together, and by preventing swelling. Permeability is improved by modifying pore size and pore size distribution.

Recent advances in soil improvements using admixtures include extended applications of lime and cement in structural fills and with deep mixing methods, and the development of new materials.

Soil Reinforcement. The most intensely studied and advanced method of soil improvement is soil reinforcement. Composites of earth with reinforcing inclusions and in-situ ground reinforcement have already been mentioned: sand and gravel compaction piles; piers, piles, and walls constructed by deep mixing method; and ground strengthening by heating or freezing. Four other types of soil reinforcement commonly used today are stone columns, root piles, soil nailing, and reinforced earth.

Stone columns are compacted columns, 0.5 to 1 meter in diameter, of crushed rock or gravel installed in soft soils. They provide vertical support for structures or embankments and increase shear resistance in horizontal and inclined directions. Stone columns also act as drains.

Root or micropiles are reinforced (usually) concrete cast-in-place piles, 0.1 to 0.25 meters in diameter. They are installed in groups of inclined and vertical piles. Root piles support structures and stabilize inclined soil.

Supporting the ground by grouting reinforcing bars into the soil is called soil nailing. The purpose is similar to root piles and is used primarily for improvement of slope stability and support of excavations.

Reinforced earth uses metals and geotextiles as reinforcements. It is a constructural composite material in which compacted fill and tensile reinforcement are built up in alternating layers. The reinforcements are used to carry tension only.

Existing Construction

Use of drains that relieve the build-up of pore pressure is the major approach applicable to sites with existing structures. There are four types of drains applicable to liquefaction mitigation: stone, sand, wick, and wells for permanent dewatering. Drains can be used in sand, silt, or clay soils. Primarily, stone or gravel drains are used to prevent liquefaction by dissipating pore water pressures nearly as quickly as they are generated in earthquake-induced cyclic loading. Analytical and experimental studies have shown gravel drains to be effective in dissipating excess pore water pressure due to their high permeability (Ref 3, 4, and 5). Sand and wick drains may be used to supplement gravel drains and to relieve existing pore water pressure in a confined layer of soil susceptible to liquefaction. Gravel and sand drains are installed vertically and used to control the not-underwater free field adjacent to a structure. When the free field is underwater these drains also have potential, but special techniques would be required. Wick drains can be installed at any angle and may be used to relieve excess pore water pressure under an existing structure as well as in the free field. Using wells for permanent dewatering is very expensive.

PRINCETON UNIVERSITY EFFECTIVE STRESS SOIL MODEL

In order to analyze the response of a large complex waterfront structure on soil in which seismically-induced pore pressures cause loss of soil stiffness and shear strength, and hence liquefaction of the soil, a constitutive soil relationship capable of predicting soil behavior under generalized loading conditions was developed at Princeton University (Ref 6). The authors demonstrated that implementing this effective stress soil model into DYNAFLOW, developed by Professor Prevost, allows a rational analysis of complex dynamic soil-structure interaction problems.

Professor J.H. Prevost of Princeton University (Ref 7) has developed a finite element numerical procedure to analyze transient phenomena in fluid saturated porous media. The saturated porous medium is modeled as a two-phase system consisting of a solid and a fluid phase. The solid skeleton may be linear, or nonlinear and hysteretic. Large deformations may also be included. The fluid may be compressible or incompressible depending upon the intended application (blast, seismic, etc.). Time integration of the resulting semidiscrete finite element equations is performed by using an implicit-explicit algorithm (Ref 8 and 9). In order to remove the time-step size restriction associated with the presence of the stiff fluid in the mixture, the fluid contribution is always treated implicitly.

STUDY PROGRAM

Once a seismic stability problem has been verified, present state of the art requires field tests be conducted to insure that an improvement method is applicable under specific site and soil conditions, and to verify that the method will perform as desired. This study will focus on the use of stone drains to prevent the excessive build-up of pore water pressure during earthquake loading, thereby reducing damage to the Navy's ocean front structures as a result of liquefaction.

Under certain conditions, addition of stone columns to a foundation can improve the ground when subjected to a strong motion earthquake. The mechanisms involve densifying the soil surrounding the stone column, increasing the shear strength of the foundation, and dissipating excess pore pressure through radial drainage to the columns. Stone columns typically replace 20 to 35 percent of the soil being improved (Ref 10).

Densification of the surrounding soil takes place during construction due to the heavy vibratory equipment used to install the columns. Stone columns constructed of coarse, open-graded stone are quite dense and not likely to liquefy. The shear strength of the stone column can significantly increase the lateral force resistance of the foundation. Stone columns on relatively close centers provide vertical drainage which greatly reduces the flow path and is effective in dissipating excess pore pressures generated by earthquake loading. If the pore water pressures generated in a soil mass by cyclic loading can be dissipated to some extent as they are created, the danger of liquefaction may be averted (Ref 4).

Construction techniques, site conditions, and the actual earthquake loading determine the contributions from each of the stone columns improvement mechanisms.

Seed and Booker (Ref 4) developed a design methodology to determine the required geometry needed to install a system of stone columns capable of dissipating pore pressures generated in sand layers due to strong motion earthquake vibrations (Figure 1). In this method a design earthquake is represented by N equivalent uniform stress cycles of a specific magnitude and period. The resulting pore pressure ratio (pore pressure/initial stress) is determined based on the horizontal permeability of the sand, duration of the earthquake, unit weight of water, coefficient of compressibility, and diameter of stone columns. The pore pressure ratio (pore water pressure divided by initial vertical stress) must be less than one to prevent liquefaction. The permeability of the drain is required to be at least 200 times greater than the soil being drained.

Extensive field evidence has shown that stone columns act effectively as drains for static load applications. The ability of stone columns to act as drains during earthquakes has not been demonstrated. The most appropriate method for validating the Seed and Booker design method would use field data from instrumented prototype situations. The nonavailability of field data preempts such a study. As an alternative, the predictive capability of the Princeton Effective Stress Soil Model (PESSM) will be utilized. The PESSM has been proven capable of capturing the generation of excess pore water pressure in saturated sand deposits during earthquakes (Ref 6).

This study is directed toward examining the predictive capabilities of the numerical procedure proposed in Reference 6. Using DYNAFLOW, the capability of stone columns to improve liquefiable soil foundations to assure the safe performance of structures founded on them in the event of earthquake excitation will be assessed. Site, soil, and acceleration conditions will be input to DYNAFLOW as well as the proposed course of action for the critical sites. DYNAFLOW will be used as a tool to determine if the mitigation techniques will reduce the risk of failure or assure that the consequences of a damaging earthquake will be tolerable.

Of particular interest is the value of the proposed numerical model in adequately predicting the generation of excess pore water pressures in saturated sand deposits beneath foundations during earthquakes, dissipation of the excess pore pressures through stone drains, and its performance in dynamic soil-structure interaction problems. The most appropriate method for such a validation study would be to utilize field data from an instrumented prototype situation. However, such a study is preempted by the scarcity of field data. An alternate method of validation is provided by comparison of results from the numerical procedures with state-of-the-art theory of gravel drainage systems to stabilize potentially liquefiable sand deposits (Ref 4).

The PESSM was validated in Reference 6 by analyzing centrifuge soil model test data. Although imperfect in many respects, it is felt that dynamic centrifuge soil model tests can provide a data base for calibration of numerical procedures. A number of dynamic centrifuge soil model tests have been reported and analyzed (Ref 11 and 12). Of particular interest to this study is the dynamic centrifuge brass footing soil

model test reported in Reference 13. Reference 6 presents results of a comparison of the numerical method incorporated in DYNAFLOW and the dynamic centrifuge tests.

STUDY PROCEDURE

The brass footing centrifuge test is the reference point by which the effectiveness of the stone columns will be measured. The soil was placed in a stacked-ring apparatus by pluviating the sand in layers into water and then rodding to achieve the desired density. A brass cylinder (diameter = 113 mm) was placed on top of the saturated sand deposit (height = 151 mm, diameter = 406 mm) and consolidated on a centrifuge at a centrifugal acceleration of 80 g's. The deposit was then subjected to sinusiodal base acceleration. The corresponding prototype situation was analyzed using DYNAFLOW.

Figure 2 shows the finite element mesh used for analysis. The soil is discretized using 240 elements and the footing by using two rows of 10 elements each. The soil parameters are given in Reference 6 and in the analysis permeability = $2.5 \times 10^{-3} \times 80$ m/sec to properly scale diffusion time. The footing is modeled as a one-phase elastic solid with mass density = 8.5×10^{3}/kg/m^{3} and Young's Modulus = 10^{12} N/m^{2}.

The water table is located at the ground surface. Drainage of the pore fluid is not allowed to take place through the rigid bottom boundary or the lateral side boundaries. Ground shaking is applied as a horizontal sinusiodal input acceleration at the bottom boundary nodes, with a maximum acceleration of 0.17 g (1.6677 m/sec^{2}) and a frequency of 1 Hz for 10 seconds (10 cycles).

The stacked-ring apparatus is used in the centrifuge test to simulate free-field conditions. Each node in the computer model was assigned four translational degrees of freedom: two for the soil skeleton and two for the fluid phase (pore water). In the free-field simulation, the nodal planes must remain horizontal and can only undergo parallel motions. This is specified by assigning the same equation number to each nodal degree of freedom on the same horizontal plane for the two side boundaries.

Reference 6 used existing centrifuge test data which required performing laboratory triaxial tests to derive the material model for input to DYNAFLOW. The specifications for the triaxial tests required that the specimens be constructed at the same relative density as the soil used in the centrifuge tests. Due to different preparation techniques, there were differences in void ratios between the respective soils. This affected the ultimate strength and moduli of the soil. Additionally, the authors of the centrifuge test report (Ref 13) acknowledged difficulties in precisely controlling the properties of the centrifuge soil deposit. These factors combined to generate a computer simulation that caused much larger strains to be generated to reach an equilibrium state of stress; however, it was only moderately weaker in ultimate strength. The difficulties encountered in experimental procedure are inconsequential to the present study.

Two finite element meshes were designed and modeled in DYNAFLOW to determine the effect of adding columns to reduce liquefaction potential.

The soil modeled is Leighton-Buzzard 120/200 sand, whose soil model was derived and reported in the brass footing test (Ref 6). Horizontal sinusiodal base accelerations reaching 0.17 g were applied to the finite element meshes. This corresponds to the accelerations applied to the referenced brass footing centrifuge test. Figure 3 shows the acceleration history input at the base nodes.

In order to determine the best modeling scheme to achieve realistic results, several parametric studies involving a variety of modeling techniques were tried. Parametric studies included the effects of variation of stone column permeability, stone column depth and load application time steps. DYNAFLOW incorporates an algorithm that "slaves" nodal degrees of freedom at different points in the finite element mesh, forcing those nodes to displace in the same manner. Modeling techniques used to simulate the interface between the sand and stone column included nodal slaving across all of the stone column, nodal slaving across part of the stone column, shared nodes, and double noding the stone column to form a contact element. These techniques are described in greater detail in the DYNAFLOW User's Manual (Ref 14).

The studies compare a homogeneous soil mass and homogeneous soil foundation to ones including a stiffer, more permeable soil which will be referred to as a stone column (Figures 2 and 4). The stone column is assigned a permeability 200 times that of the soil and a friction angle 25 percent higher. The first study compared a homogeneous soil mass (MASS-H) to a soil mass containing a stone column (MASS-SC). The second study compared the footing on a homogeneous soil foundation (FOUNDATION-H) to a foundation including four stone columns, two on each side of the footing (FOUNDATION-CS) (Figure 2). The water table is at the surface in all cases. DYNAFLOW has the capability to place the water table below the surface by defining the elements above the water table as dry and below the water table as saturated. Consolidation taking place due to seismic loading can cause a change in the level of the water table. Within DYNAFLOW, this cannot be reflected in the finite element mesh by causing dry elements to become partially or fully saturated.

Horizontal and vertical effective stresses, shear stresses, and pore pressures were recorded for the elements designated in Figures 5a and 6a. Nodal displacements, velocities, and accelerations were recorded for the nodes designated in Figures 5b and 6b.

FINITE ELEMENT STUDIES

Soil Masses

The control column (MASS-H) was designed with depth and element size comparable to the footing model. The column is made up of 1 by 1 meter elements, five wide and twelve deep (Figure 4a). Gravity forces were applied by consolidating MASS-H to 1.0 g vertical acceleration over a period of 10,000 seconds. This consolidation period was sufficient to ensure no excessive pore pressure build-up, verified by monitoring pore pressure and effective stresses within selected elements for 20,000 seconds. When consolidation was complete, sinusiodal horizontal base acceleration of 0.17 g was applied.

MASS-SC is the same size (5 by 12 meters) as MASS-H, but the center column of elements is replaced with stiffer, more permeable stone column elements (Figure 4b). MASS-SC was consolidated and shaken in the same manner as MASS-H.

Results.

Effective Stress. Both soil masses experienced a decrease in effective vertical stress when shaken. The outer elements of the two masses behaved similarly (elements 7 through 10, Figure 6a). Elements adjacent to the stone column in MASS-SC (elements 3 through 6, Figure 6a) experienced a greater decrease in effective vertical stress than the corresponding elements in MASS-H, though the amplitude of the higher frequencies was smaller in MASS-SC than in MASS-H (Figure 7).

The elements within the stone column in MASS-SC (elements 1 and 2, Figure 6a) initially experienced a decrease in effective vertical stress, but began to increase after the first second of shaking (Figure 8). When shaking ended after 10 seconds, the final effective vertical stress within the stone column elements was greater than the initial value.

Pore Pressure. Pore pressure increase due to shaking was similar in the two masses for the first 1 to 2 seconds. After 2 seconds, MASS-SC dissipated excess pore water pressure while MASS-H continued to build up excess pore water pressure. The amplitude of the higher frequencies in MASS-H were larger than those of MASS-SC, in many elements two to three times larger (Figures 9 and 10).

Summary. It is of great value to see the limited zone affected by the stone column. The addition of a stone column to a homogeneous soil mass allowed a redistribution of stress from the nearby sand (one stone column diameter away) to the stone as the sand began to lose strength, but did not affect the stress in the sand a distance of two stone column diameters away. The stone column was also shown to be effective in reducing the amount of pore pressure build-up due to seismic loading.

Footing

The second series of finite element meshes were designed to model foundations with surface footings. One mesh modeled a homogeneous soil foundation (FOUNDATION-H) (Figure 2a) and the other a soil and stone column foundation (FOUNDATION-SC) (Figure 2b). The foundations are 32.5 by 12.1 meters and the footing 9.0 by 1.56 meters. The finite element models were consolidated to 1.0 g vertical acceleration over a period of 10,000 seconds to simulate gravity forces. A static pressure of 1.30 by 10^5 N/m^2 was then applied to the top of the footing. The consolidation period was sufficient to ensure no excessive pore pressure build-up, verified by monitoring pore pressure and effective stresses within selected elements for 20,000 seconds. When gravity loading was complete, the mesh was subjected to a sinusiodal base acceleration of 0.17 g at 1 Hz for 10 seconds (Figure 3).

Results.

Effective Stress. FOUNDATION-H (homogeneous foundation) experienced large zones of liquefaction in the free field as well as some localized liquefaction beneath the structure. Great improvement was achieved with the addition of stone columns, though FOUNDATION-SC experienced some localized liquefaction (Figure 11). The majority of elements monitored showed improvement when stone columns were added. In FOUNDATION-SC, both sand and stone elements experienced an increase of up to 100 percent of the initial effective stress by the end of shaking (Figures 12, 13, and 14). Elements that liquefied in both models showed the decrease in strength to be more gradual in FOUNDATION-SC than those in FOUNDATION-H and with smaller amplitude of the higher frequencies.

Pore Pressure. Pore pressures were generated within FOUNDATION-SC during the first second of shaking, though generally to a lesser degree than in FOUNDATION-H. After the first second of shaking, excess pore pressures dissipated and, in general, the pore pressure within an element at the end of shaking was equal to or lower than the initial pore water pressure (Figures 15b, 16b, and 17b). The influence of the stone column on pore water pressure was found to affect elements two stone column diameters away.

Summary. Analysis of the effective stress time histories during shaking indicates a redistribution of the footing load to the stiff stone columns from the weakening sand. The addition of the stone columns to the sand foundation also proved beneficial in dissipating pore water pressure generated. The zone of influence of the stone columns was larger in the soil/structure situation than in the soil masses.

SUMMARY AND CONCLUSIONS

The Navy has $25 billion worth of facility investments in areas susceptible to damage from seismically-induced soil liquefaction. Liquefaction plays a predominant role in waterfront damage, often being the single cause of widespread losses. Stone columns inserted into potentially liquefiable soil increase the overall shear strength of the foundation and reduce the radial drainage path, which can be effective in dissipating excess pore water pressures generated by earthquake loading.

The Princeton University Effective Stress Soil Model, incorporated into the finite element program DYNAFLOW, has the ability to predict the generation of pore water pressure in saturated sand deposits beneath surface footings during earthquakes. Using DYNAFLOW, two finite element studies were conducted to determine the effectiveness of the inclusion of stone columns in saturated soil to improve site response when seismically loaded. In both studies, the addition of stone columns improved soil conditions and decreased damage due to the vibratory motion. The stone columns appear to stiffen up the foundation allowing for additional resistance to lateral loading from earthquake waves. Additionally, when permeable stone columns are present, less seismically-induced pore water pressure is generated due to dissipation through the stone drains.

The numerical procedure shows the predominant mechanism for improvement is the stiffening of the ground. The ability to dissipate seismically-induced pore water pressure is a secondary benefit. Experts in academia and industry agree that stiffening of the soil is the major mode of improvement.

In order to determine site response to an earthquake, it is important to be able to accurately model soil conditions. The construction of stone columns has a great affect on the conditions of the surrounding soil, causing densification of the sand and smear between the sand and stone column due to infiltration of the small sand grains into the voids of the stone column. Smear will change the permeability of the stone column as well as its effective diameter and its radial drainage area. This becomes a complex modeling problem when the numerical model has extensive analytical capabilities because it requires detailed input. Aside from the design and analysis of the stone column system, construction will be very expensive due to the large amount of stone necessary to replace the soil (20 to 35 percent). Since stone columns are installed vertically, they can only be put around, and not beneath, existing structures. Stone columns may or may not be of benefit if there are localized liquefaction zones beneath the structure. For these reasons generic improvement guidelines are inappropriate. Individual analysis of candidate sites is necessary. This effort concludes our investigation of liquefaction mitigation technology (MS-5) of the Seismic Hazard Mitigation Task.

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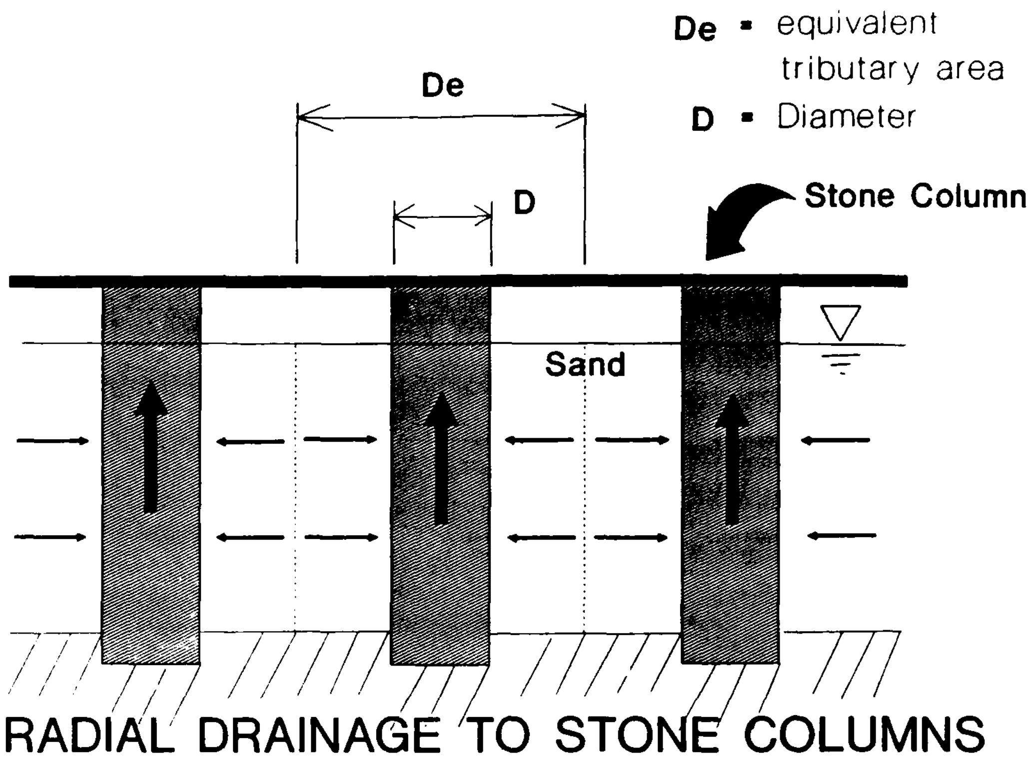
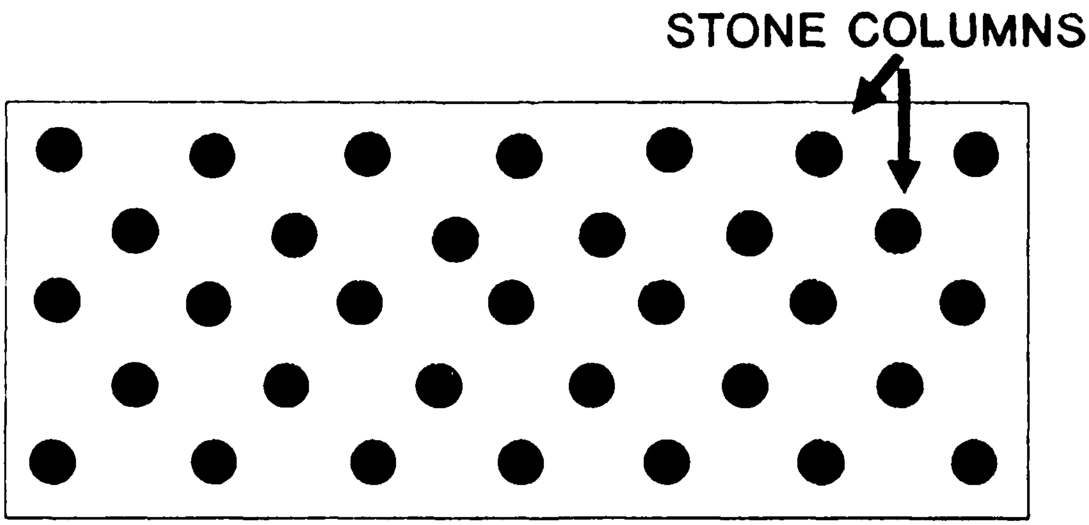
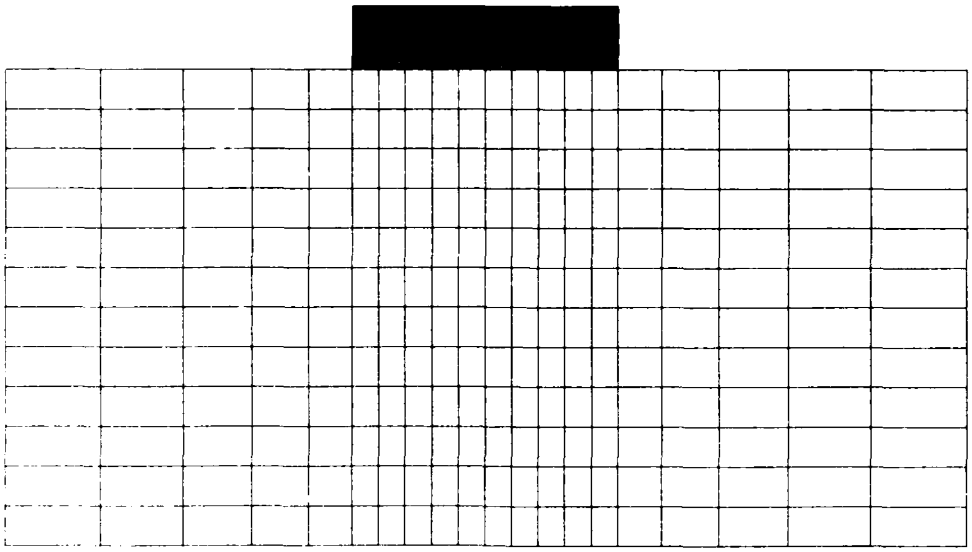


(a) Stone column drainage scheme.

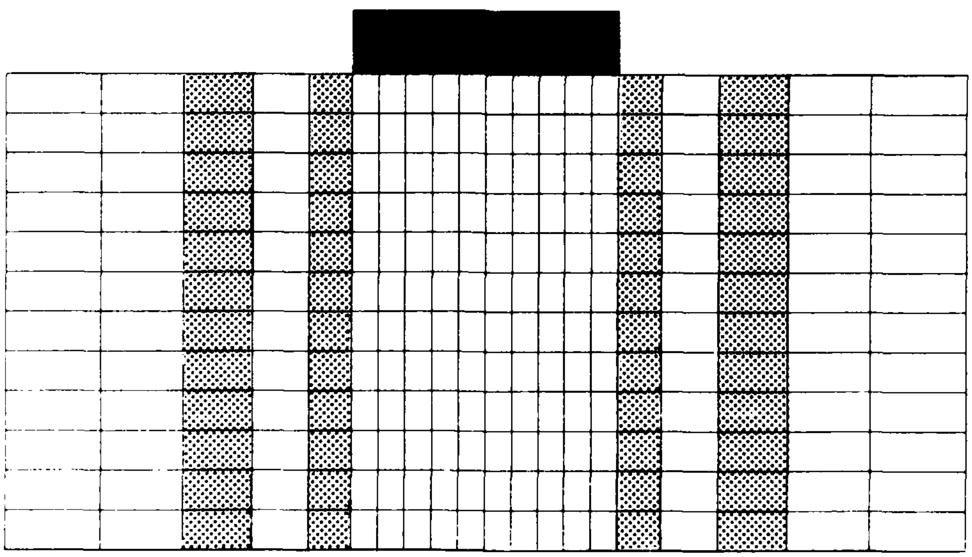


(b) Plan view of stone columns for site improvement. Generally, 20 to 35 percent soil replacement is necessary.

Figure 1. Stone column geometry.



(a) Homogeneous soil foundation: FOUNDATION-H.



(b) Soil and stone column foundation: FOUNDATION-SC.

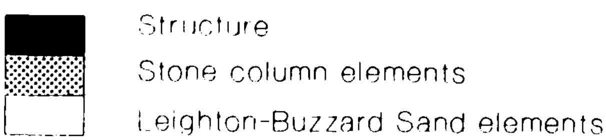


Figure 2. Foundation finite element meshes.

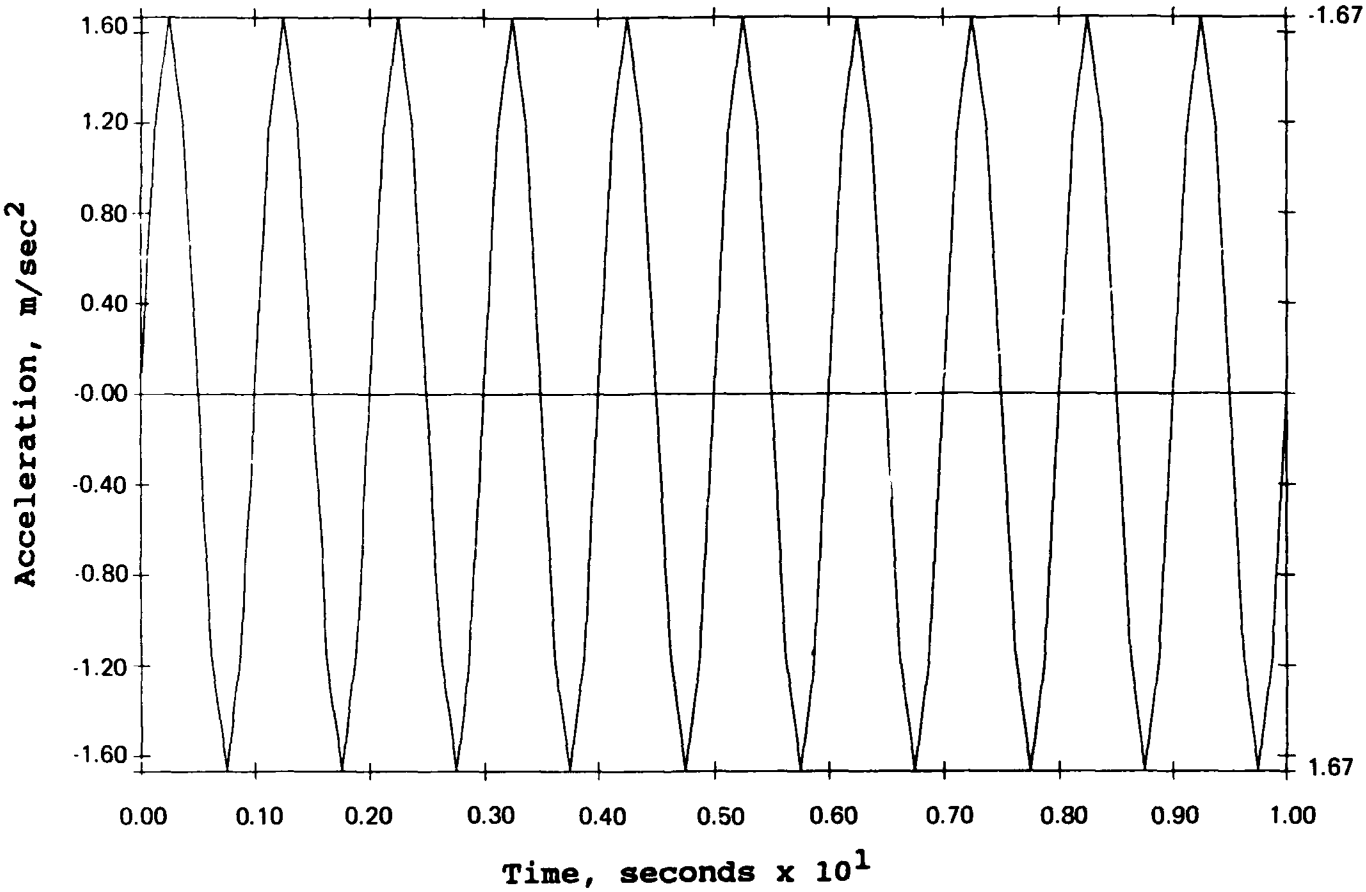
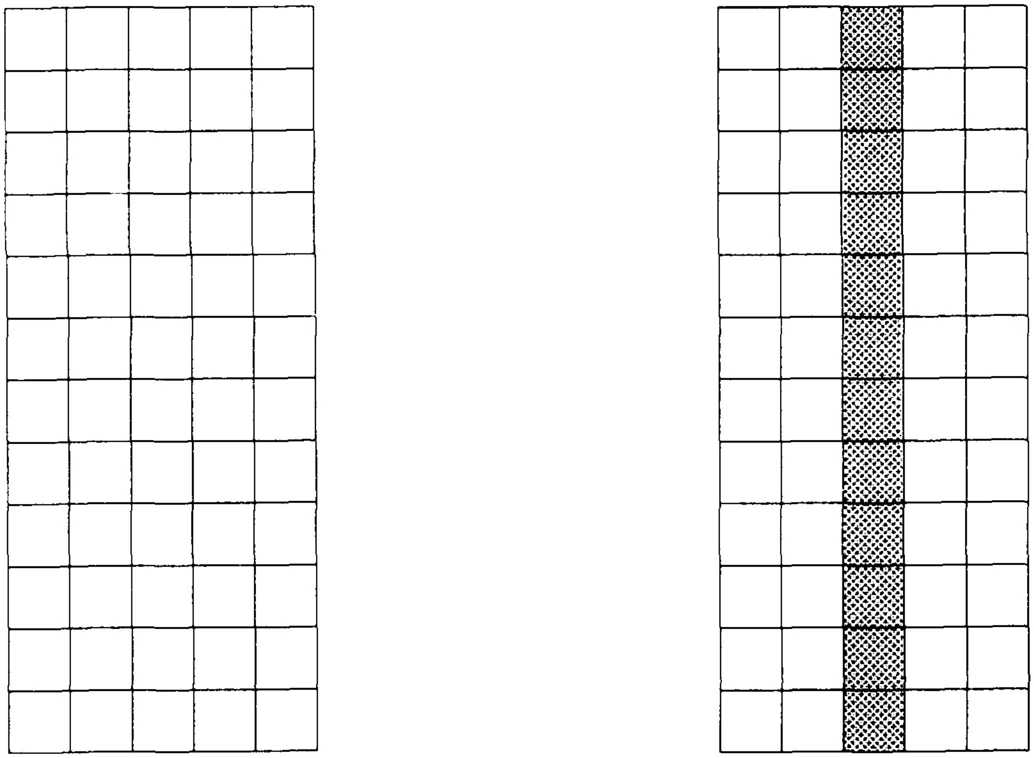
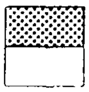


Figure 3. Sinusoidal base input acceleration.



(a) Homogeneous soil mass: MASS-H.

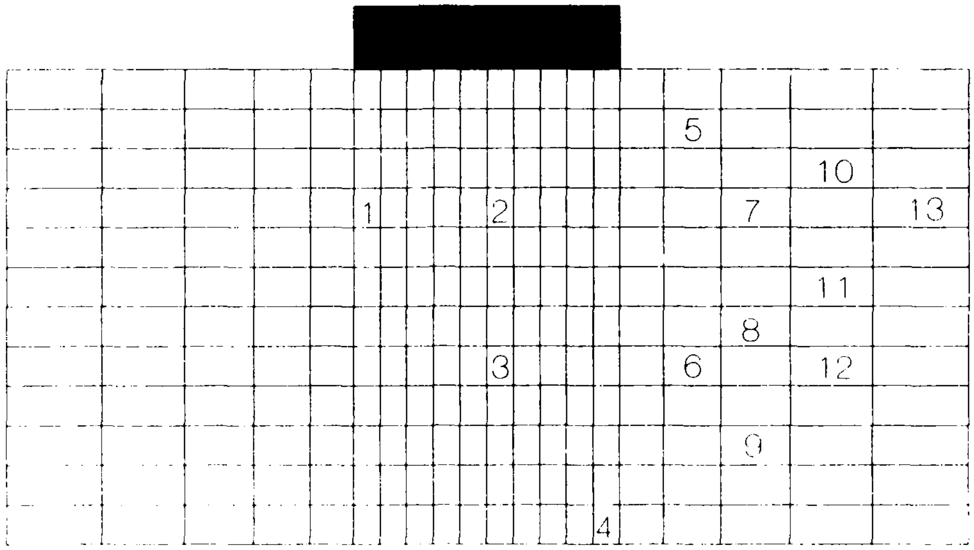
(b) Soil mass with stone column: MASS-SC.



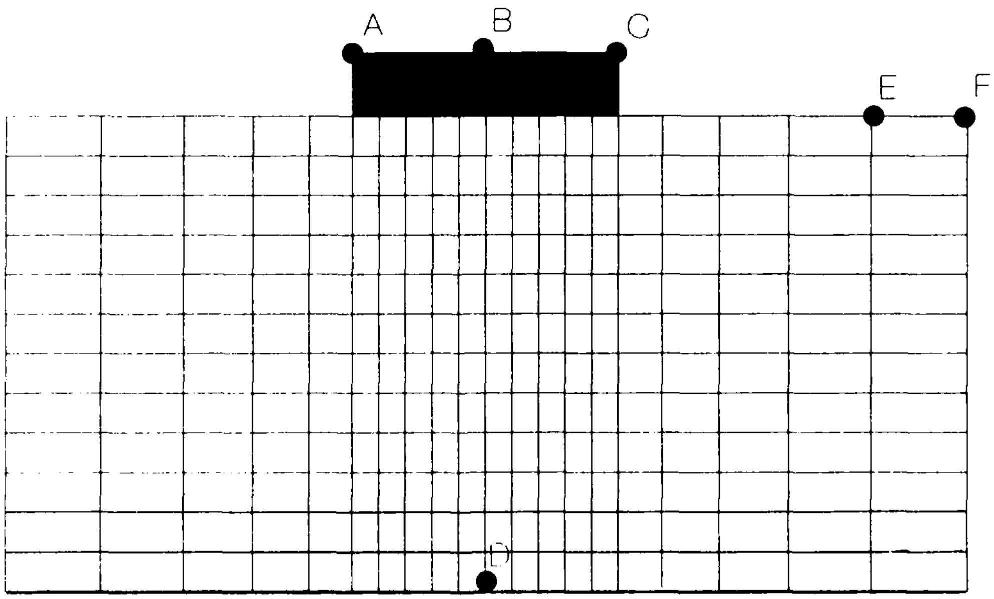
Stone column elements

Leighton-Buzzard Sand elements

Figure 4. Soil mass finite element meshes.

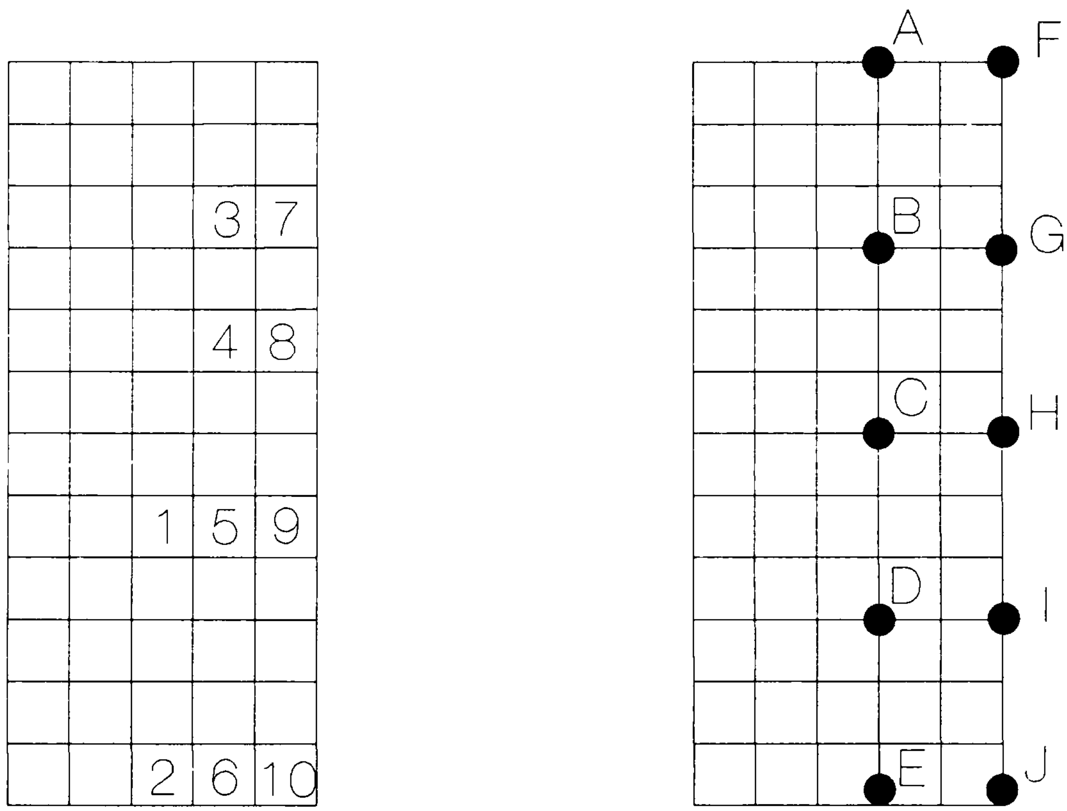


(a) Elements monitored for effective stress, shear stress, and pore pressure.



(b) Nodes monitored for displacement, velocity, and acceleration.

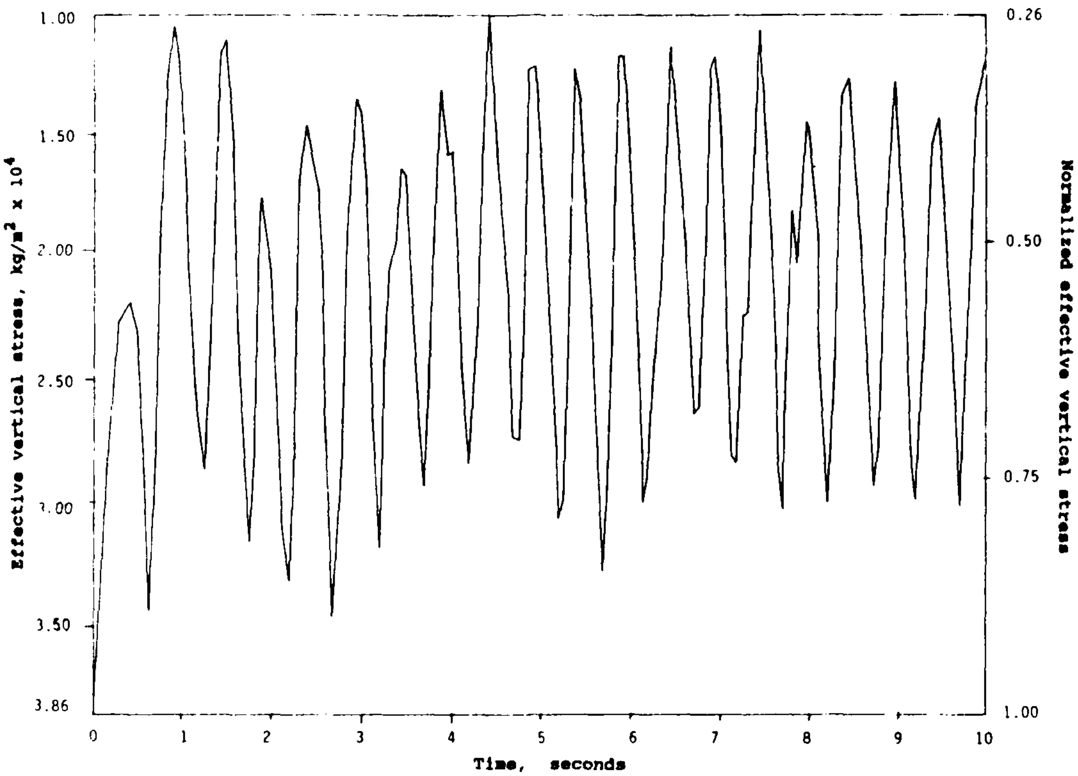
Figure 5. Foundation finite elements and nodes monitored.



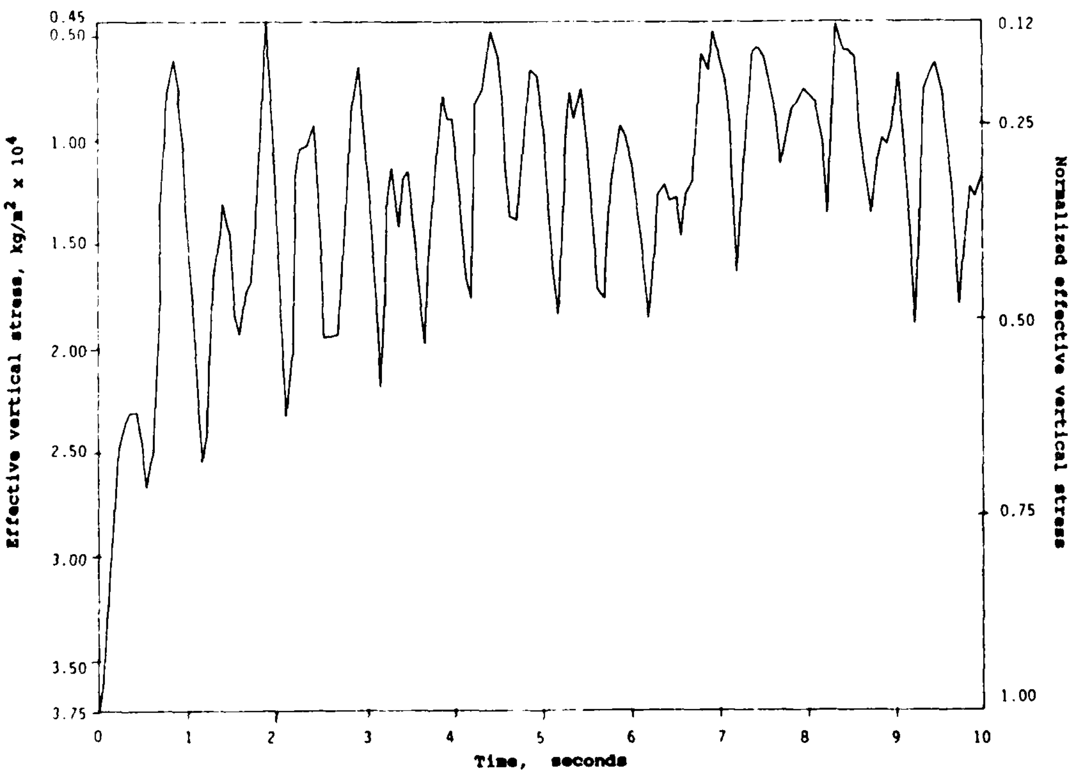
(a) Elements monitored for effective stress, shear stress, and pore pressure.

(b) Nodes monitored for displacement, velocity, and acceleration.

Figure 6. Soil mass finite elements and nodes monitored.

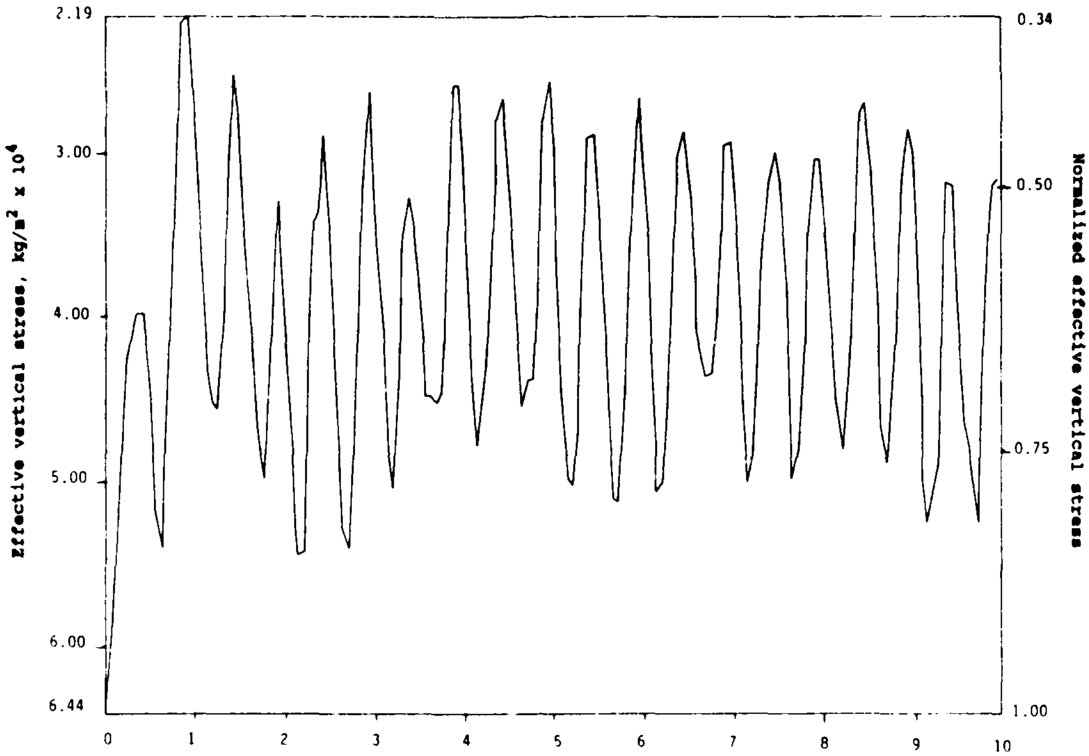


(a) Homogeneous soil mass: MASS-H.

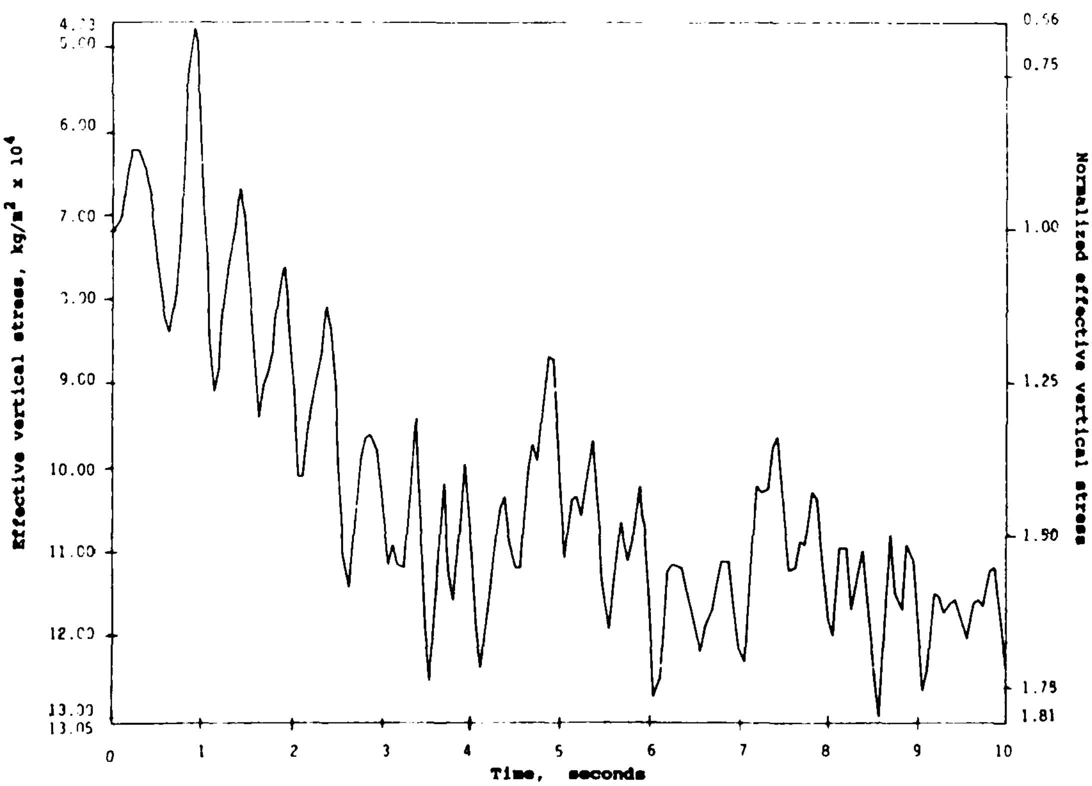


(b) Soil mass with stone column: MASS-SC.

Figure 7. Effective vertical stress in soil mass element 4.

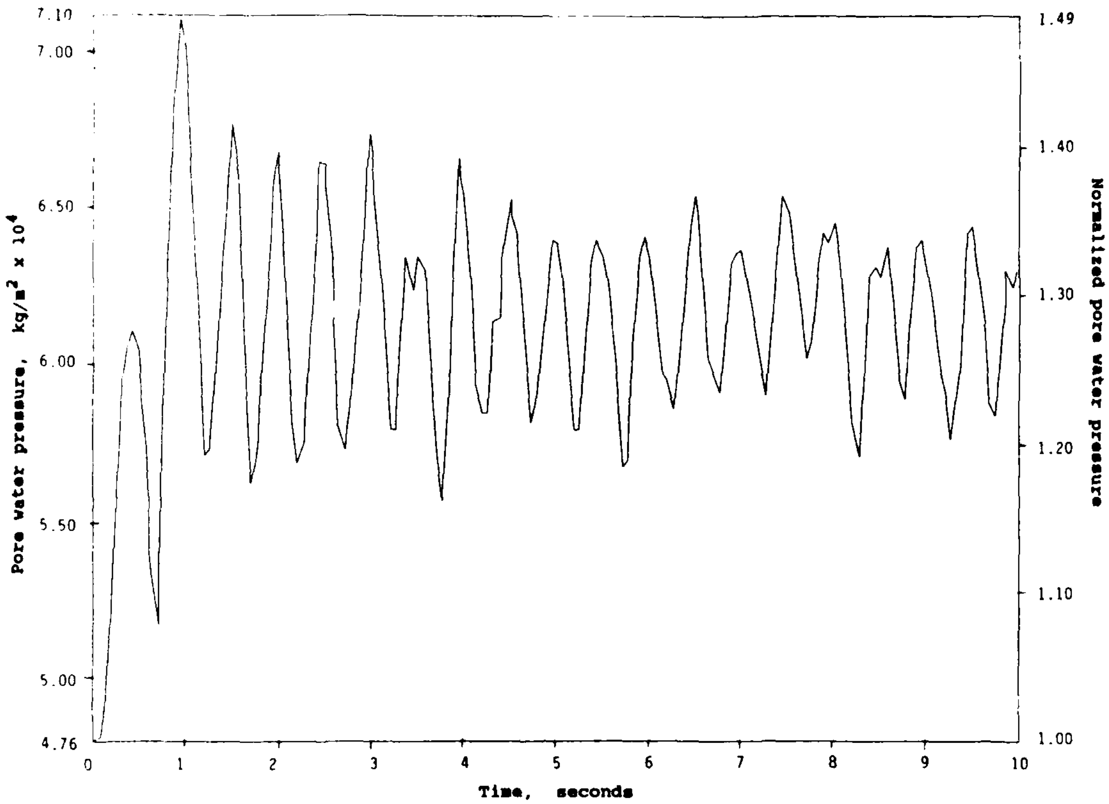


(a) Homogeneous soil mass: MASS-H.

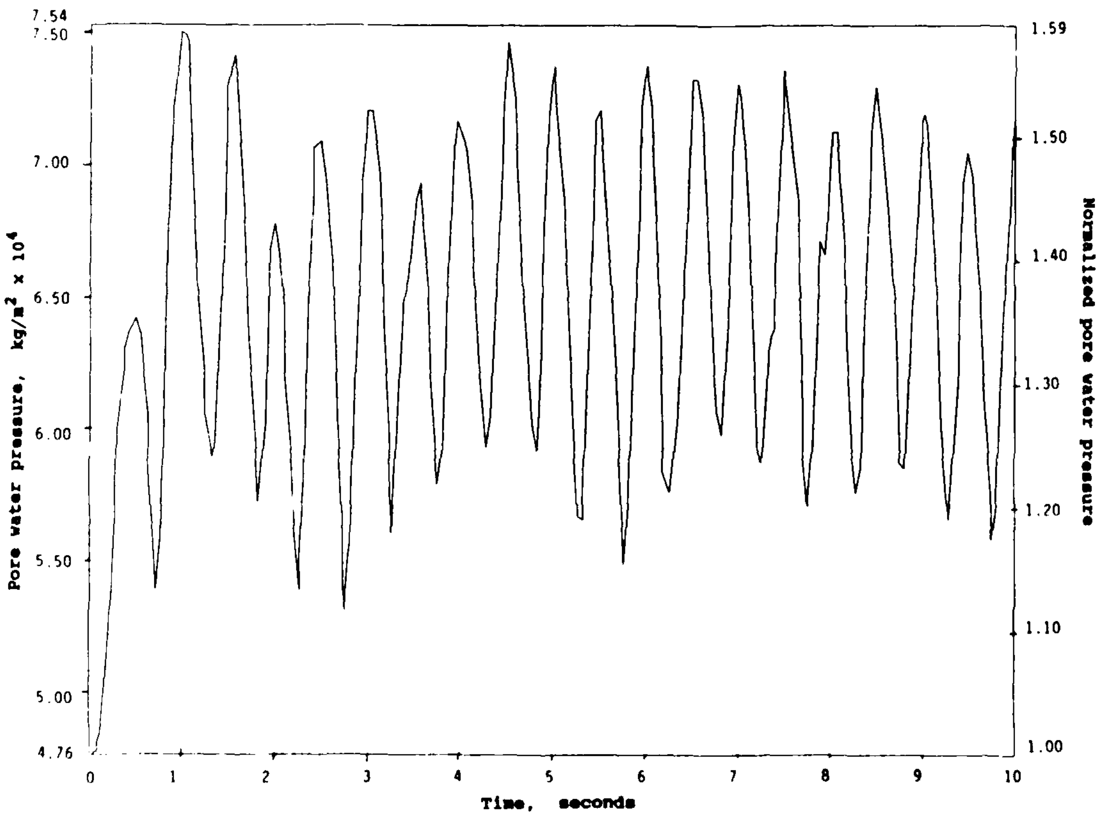


(b) Soil mass with stone column: MASS-SC.
Note: This element is in the stone column.

Figure 8. Effective vertical stress in soil mass element 1.

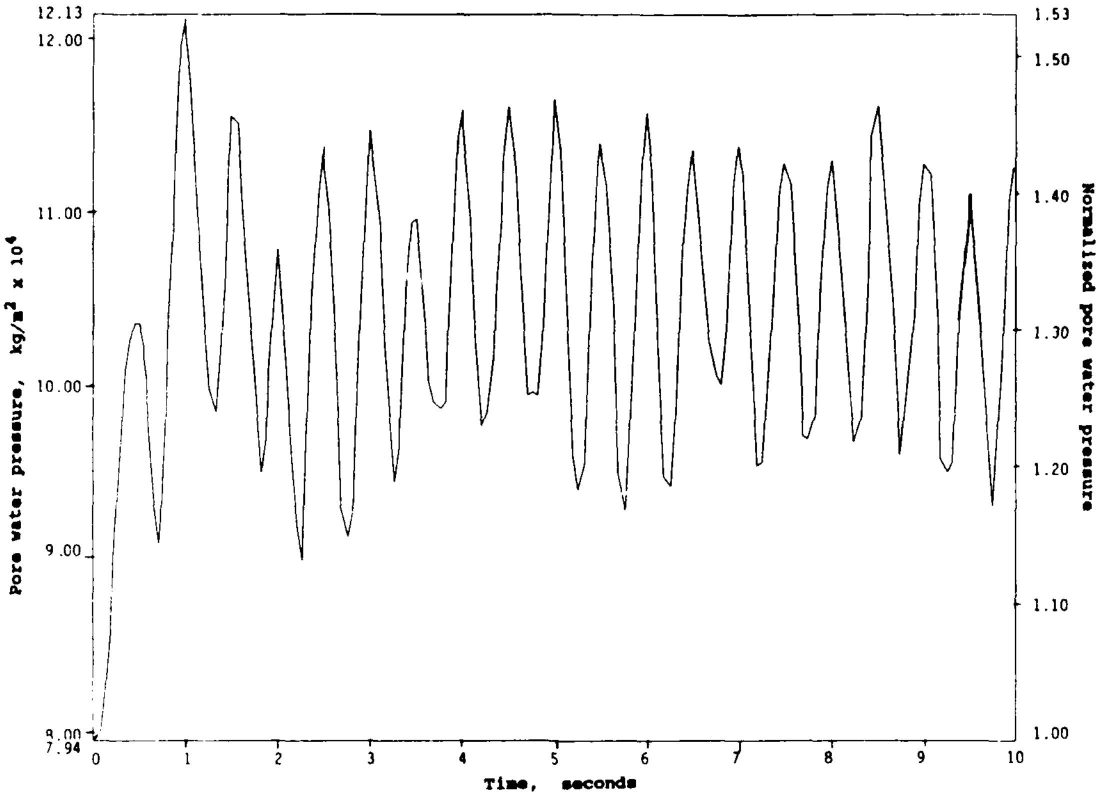


(a) Homogeneous soil mass: MASS-H.

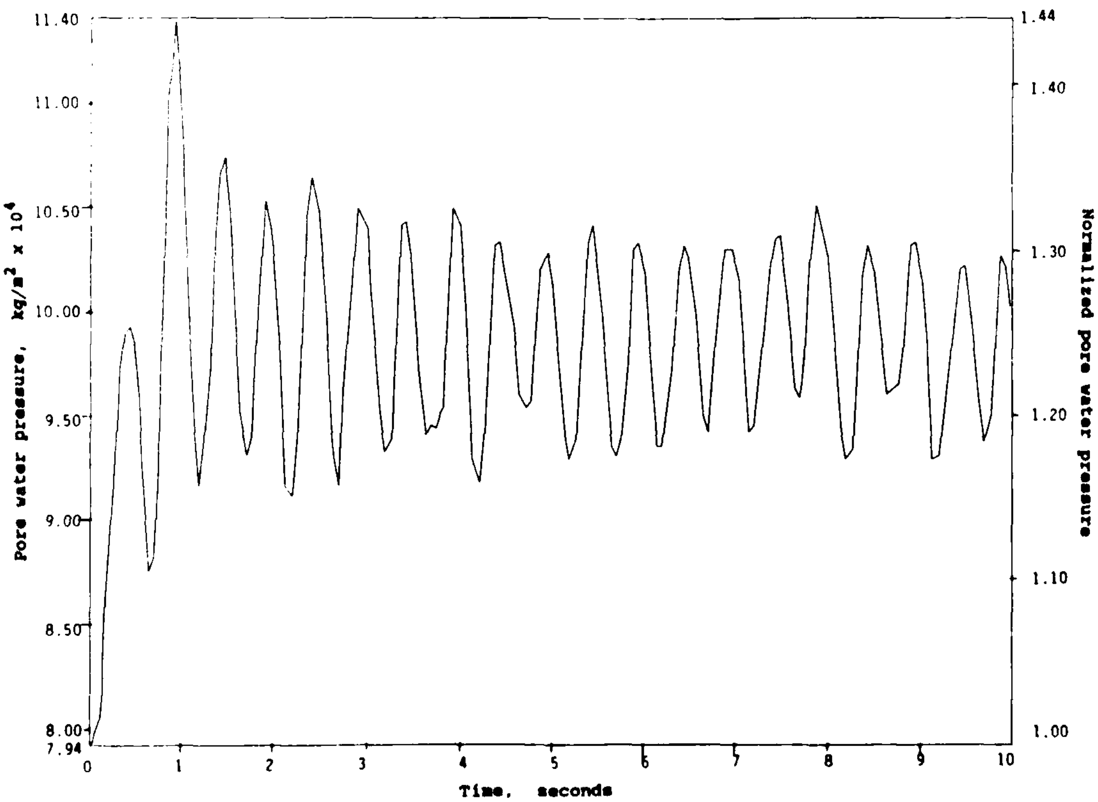


(b) Soil mass with stone column: MASS-SC.

Figure 9. Pore water pressure in soil mass element 4.

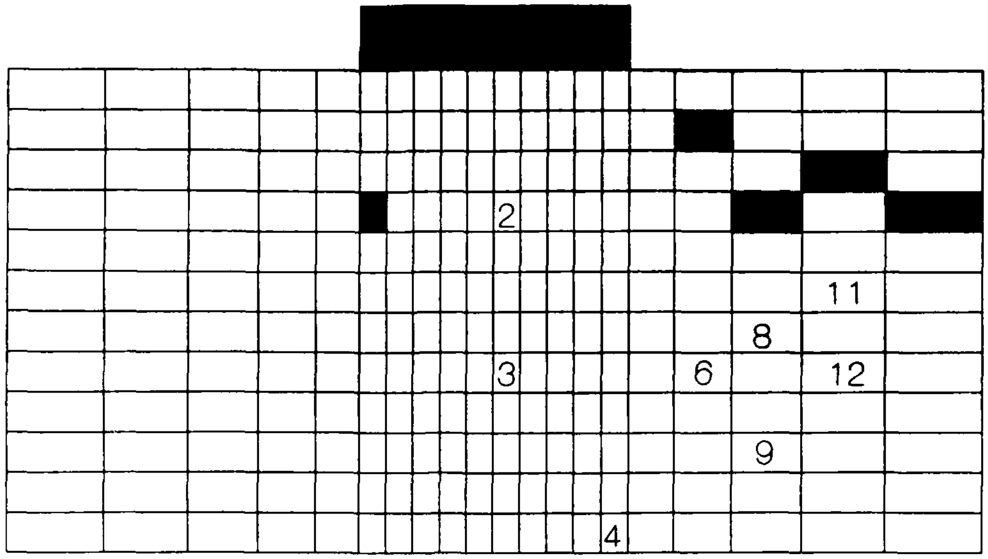


(a) Homogeneous soil mass: MASS-H.

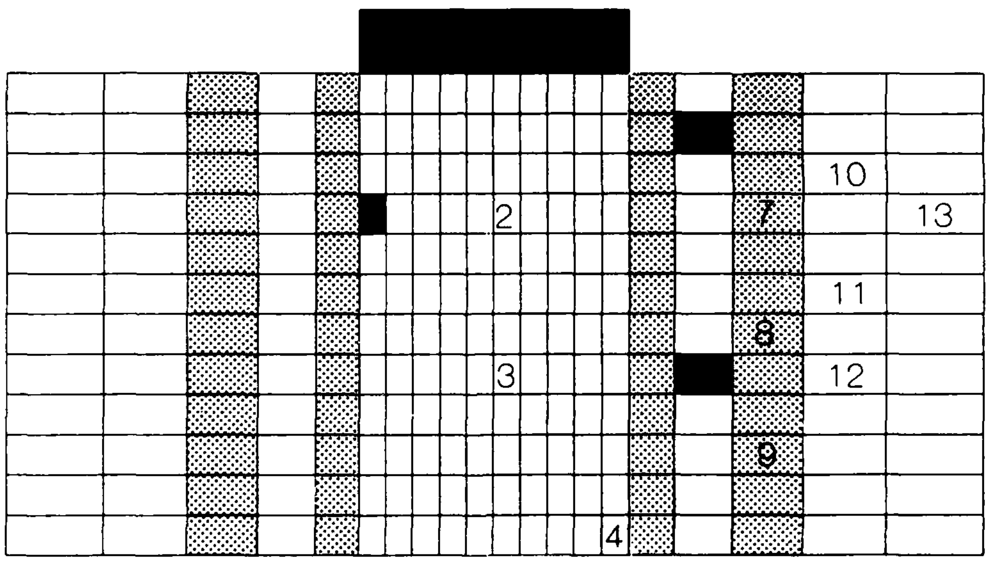


(b) Soil mass with stone column: MASS-SC.
NOTE: This element is in the stone column.

Figure 10. Pore water pressure in soil mass element 1.



(a) Homogeneous soil foundation: FOUNDATION-H.



(b) Soil and stone column foundation: FOUNDATION-SC.

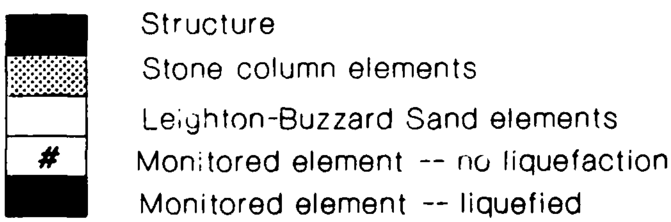
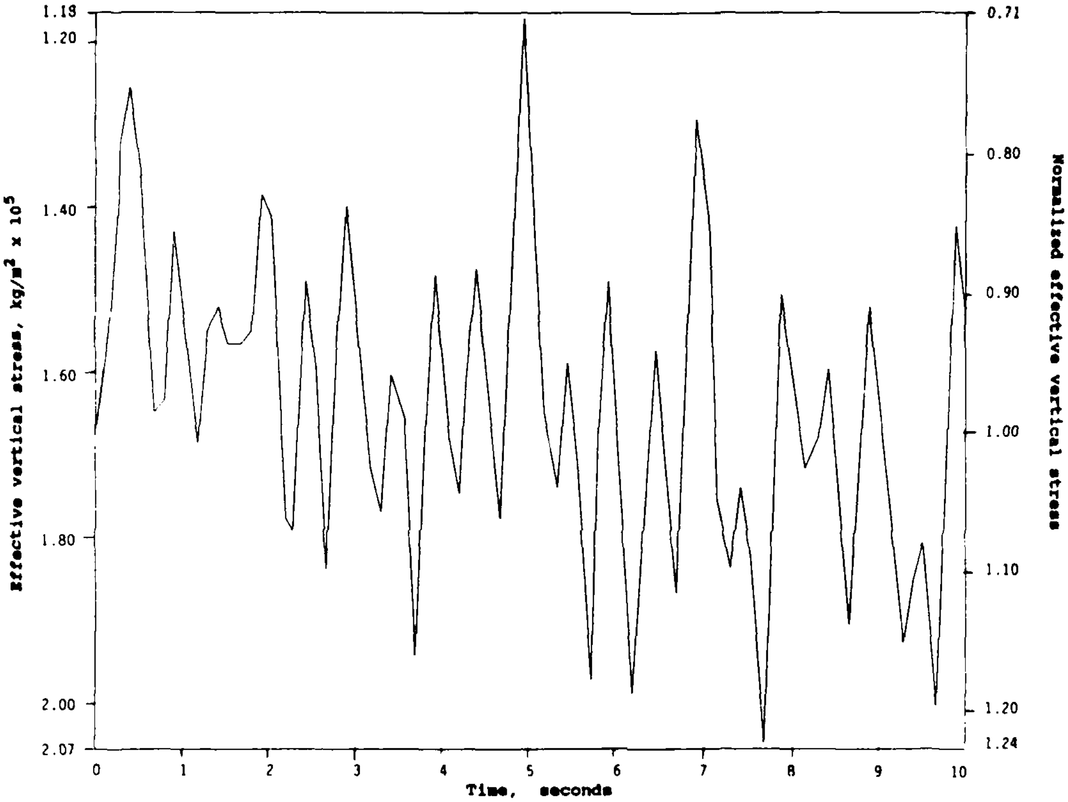
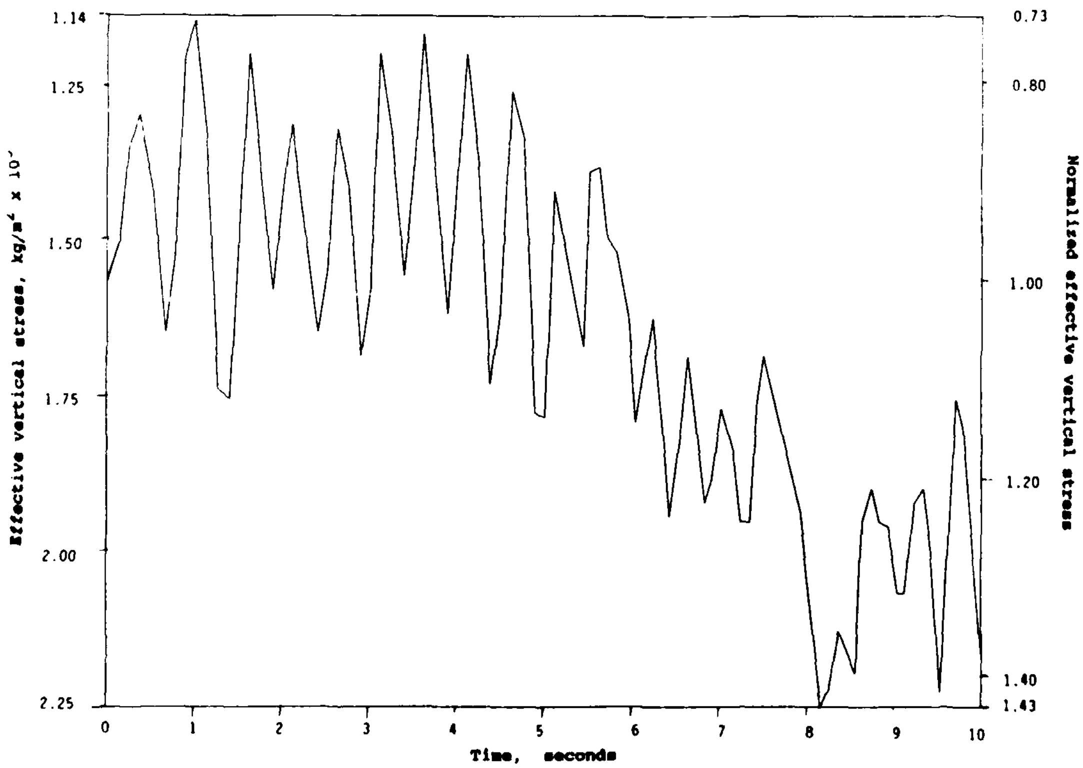


Figure 11. Elements within the footing foundation experiencing liquefaction.

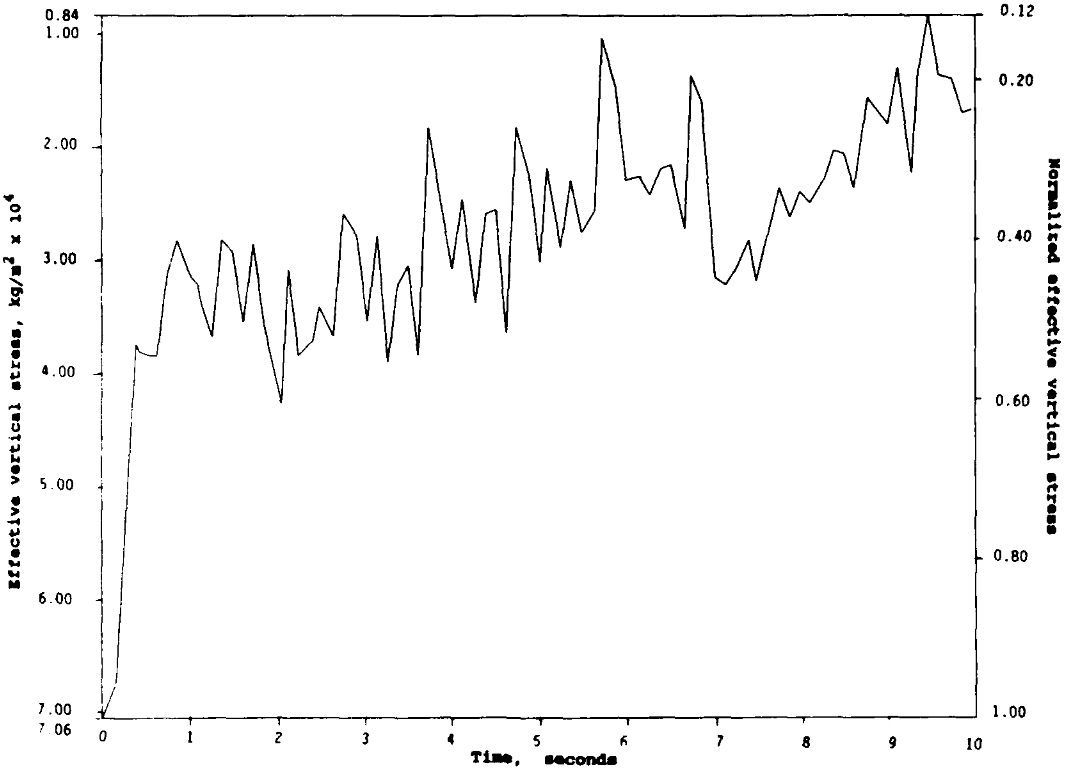


(a) Homogeneous soil foundation: FOUNDATION-H.

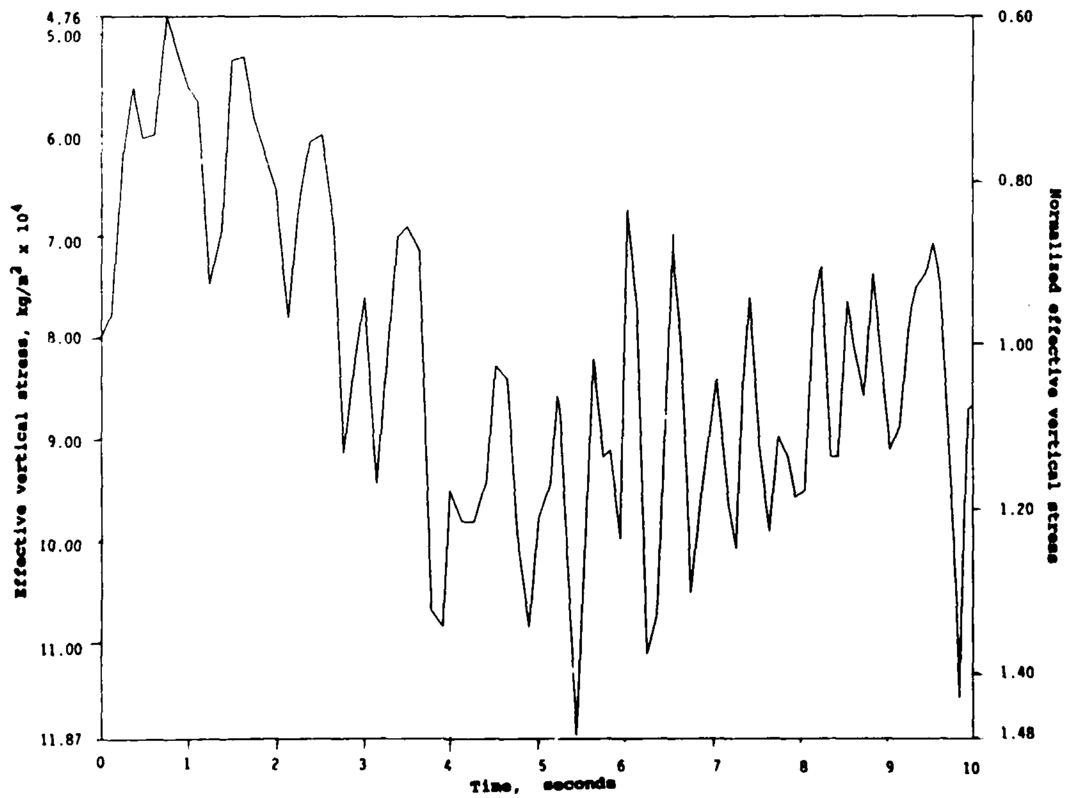


(b) Soil and stone column foundation: FOUNDATION-SC.

Figure 12. Effective vertical stress in foundation element 3.

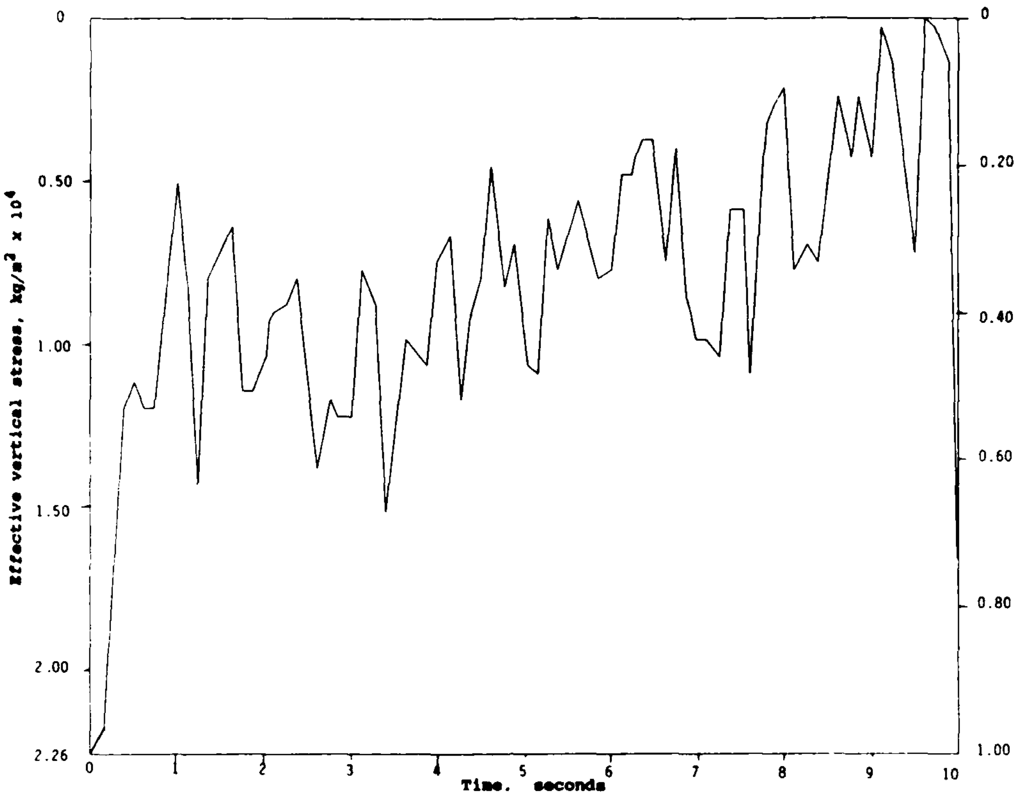


(a) Homogeneous soil foundation: FOUNDATION-H.

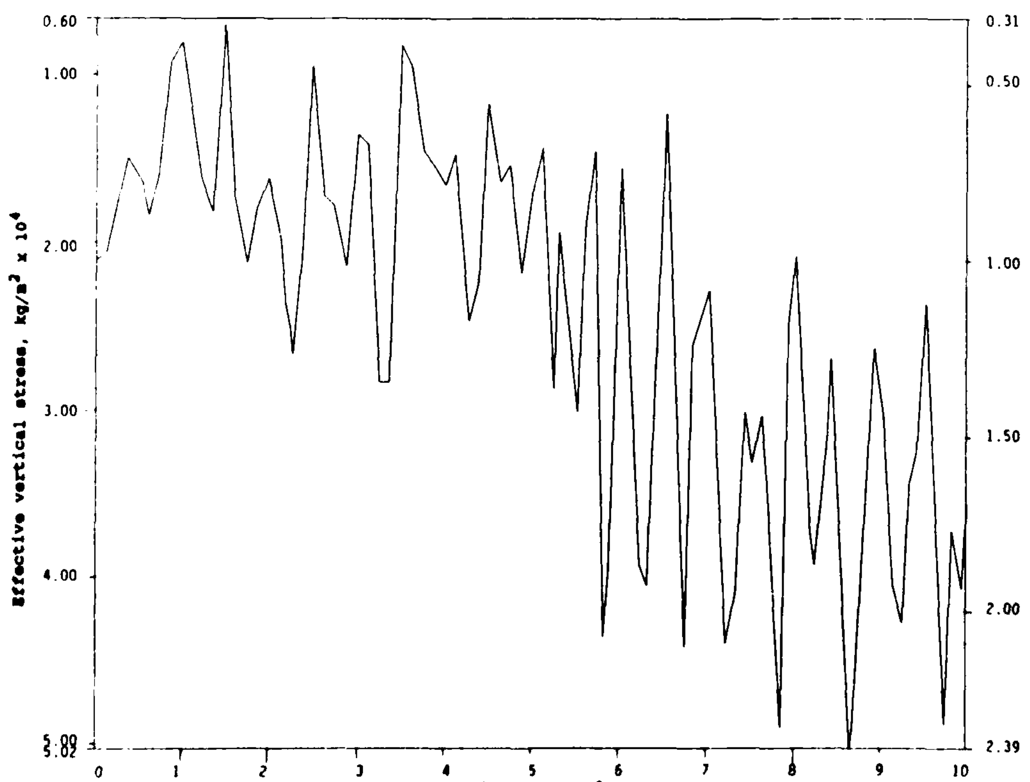


(b) Soil and stone column foundation: FOUNDATION-SC.
NOTE: This element is in the stone column.

Figure 13. Effective vertical stress in foundation element 8.

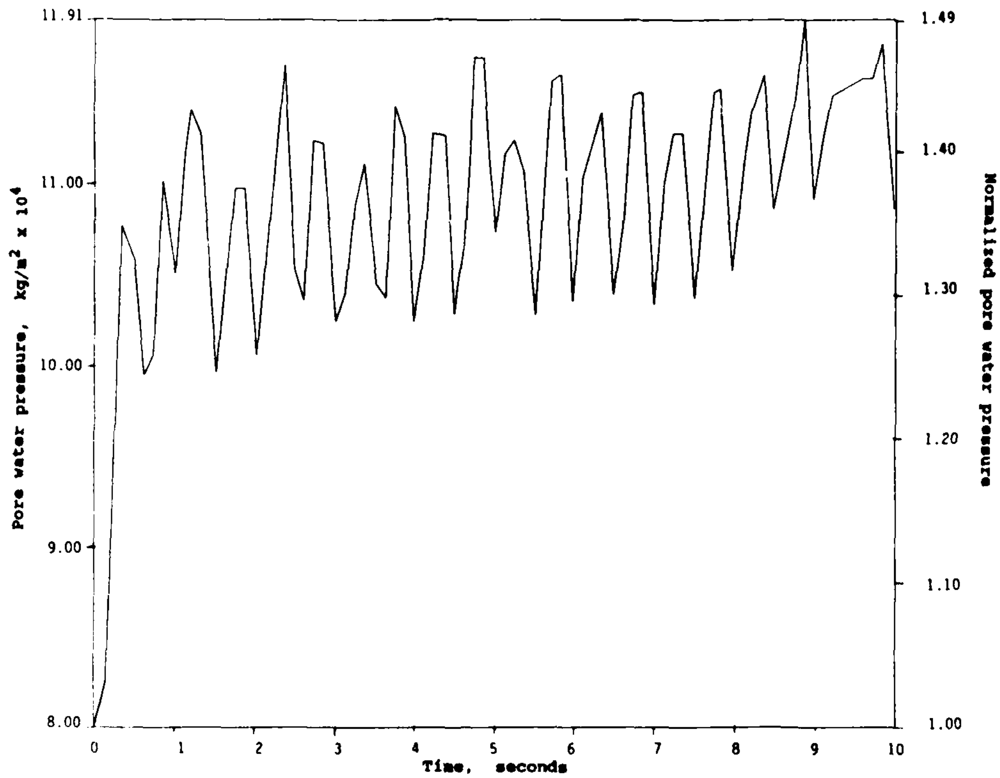


(a) Homogeneous soil foundation: FOUNDATION-H.

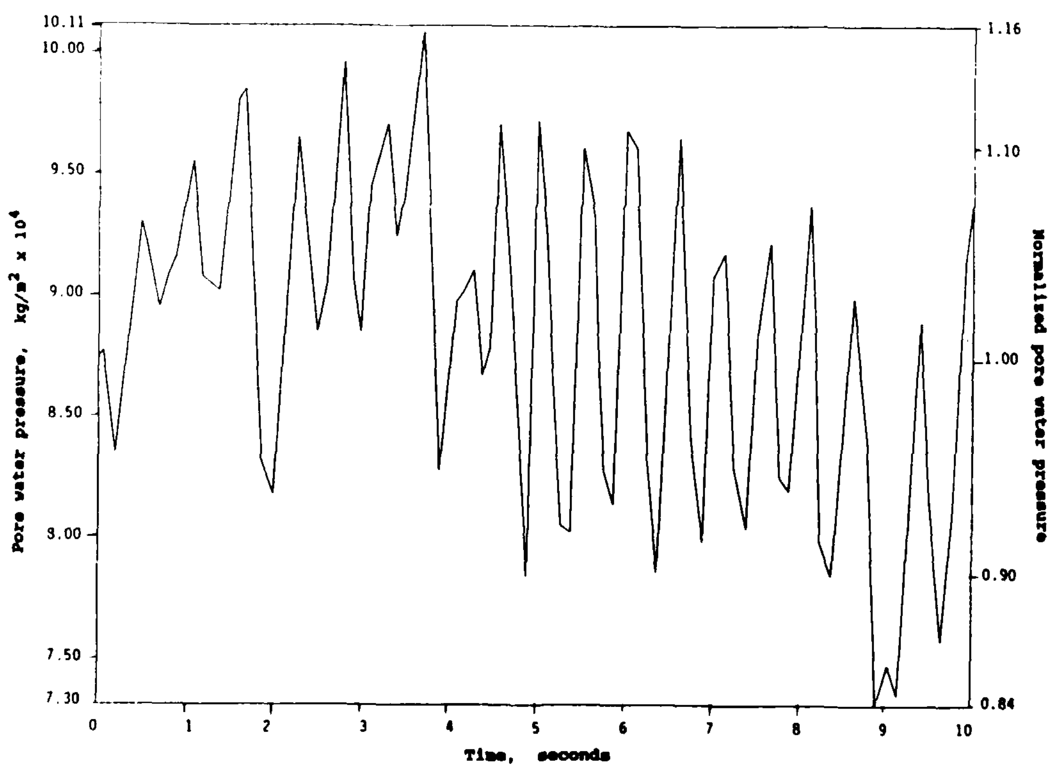


(b) Soil and stone column foundation: FOUNDATION-SC.

Figure 14. Effective vertical stress in foundation element 10.

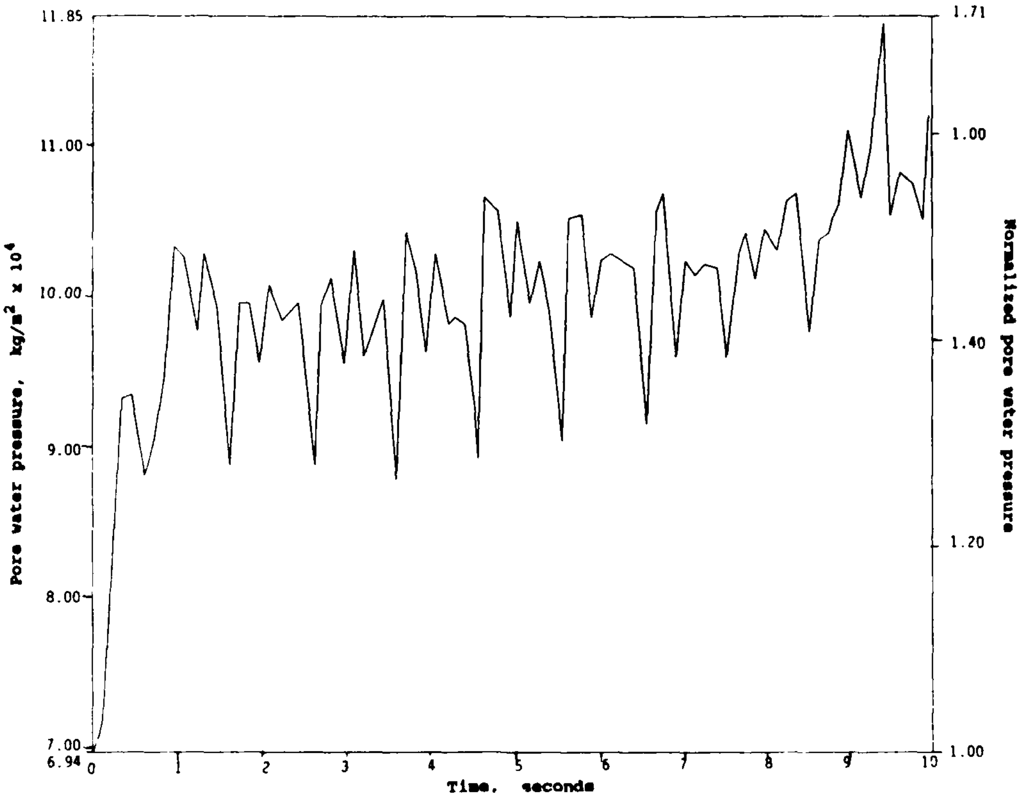


(a) Homogeneous soil foundation: FOUNDATION-H.

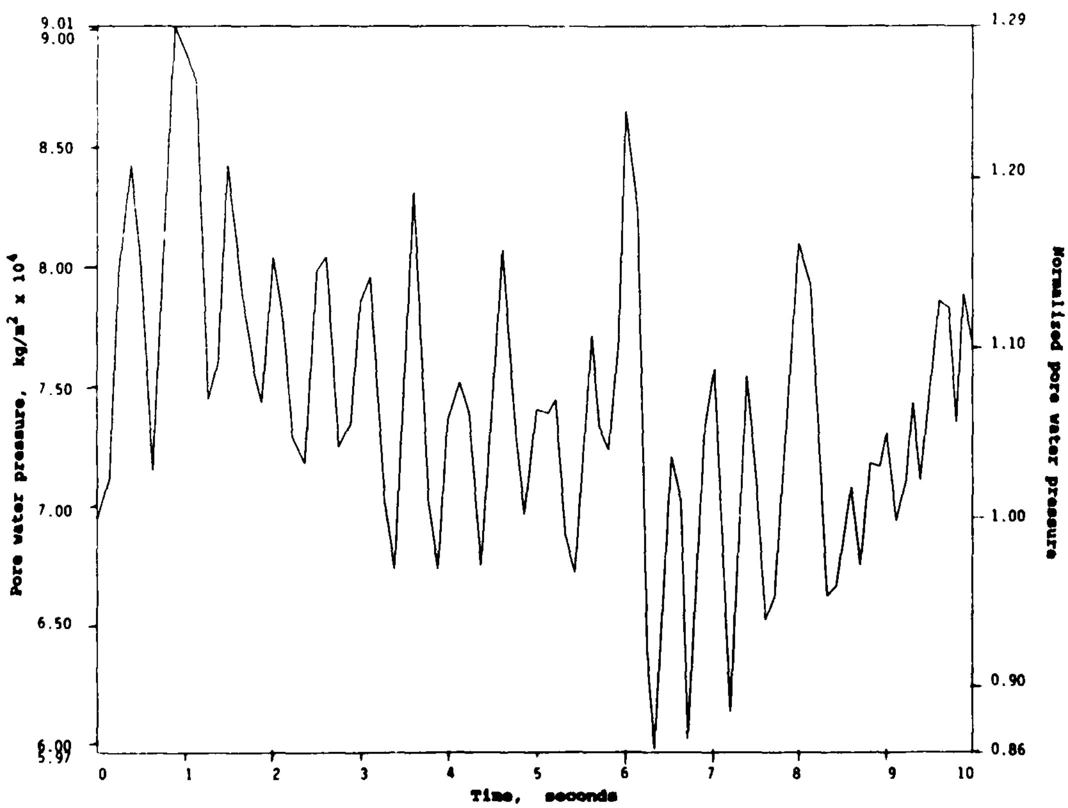


(b) Soil and stone column foundation: FOUNDATION-SC.

Figure 15. Pore water pressure in foundation element 3.

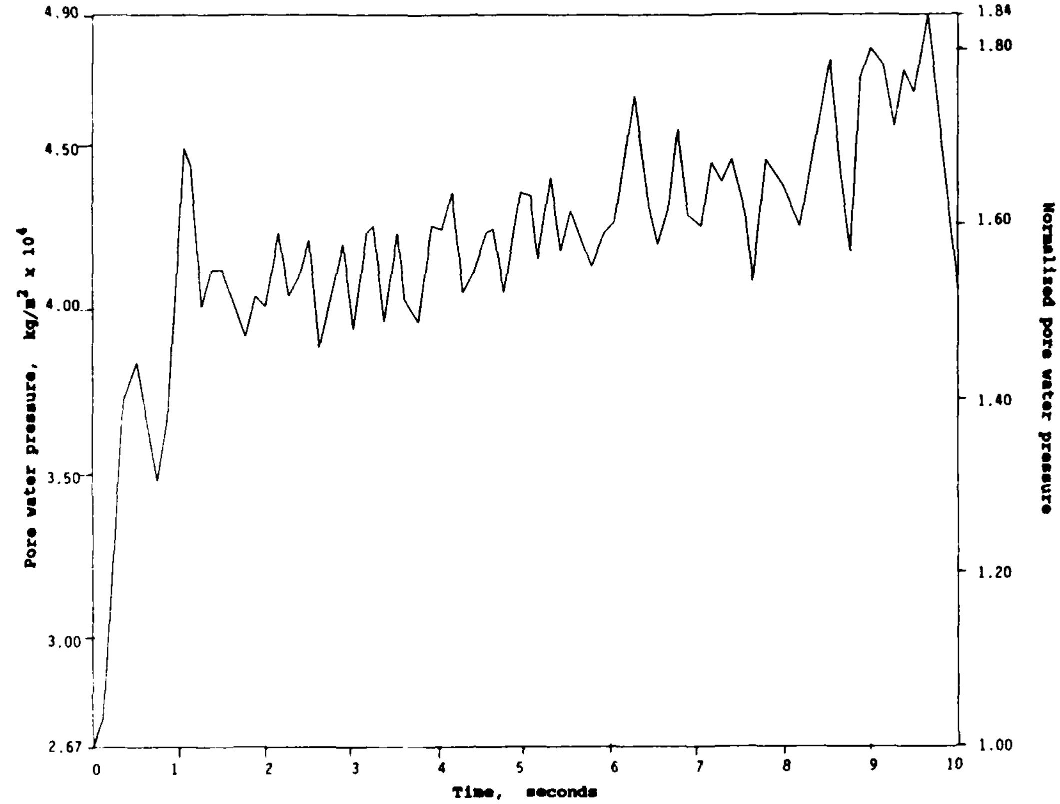


(a) Homogeneous soil foundation: FOUNDATION-H.

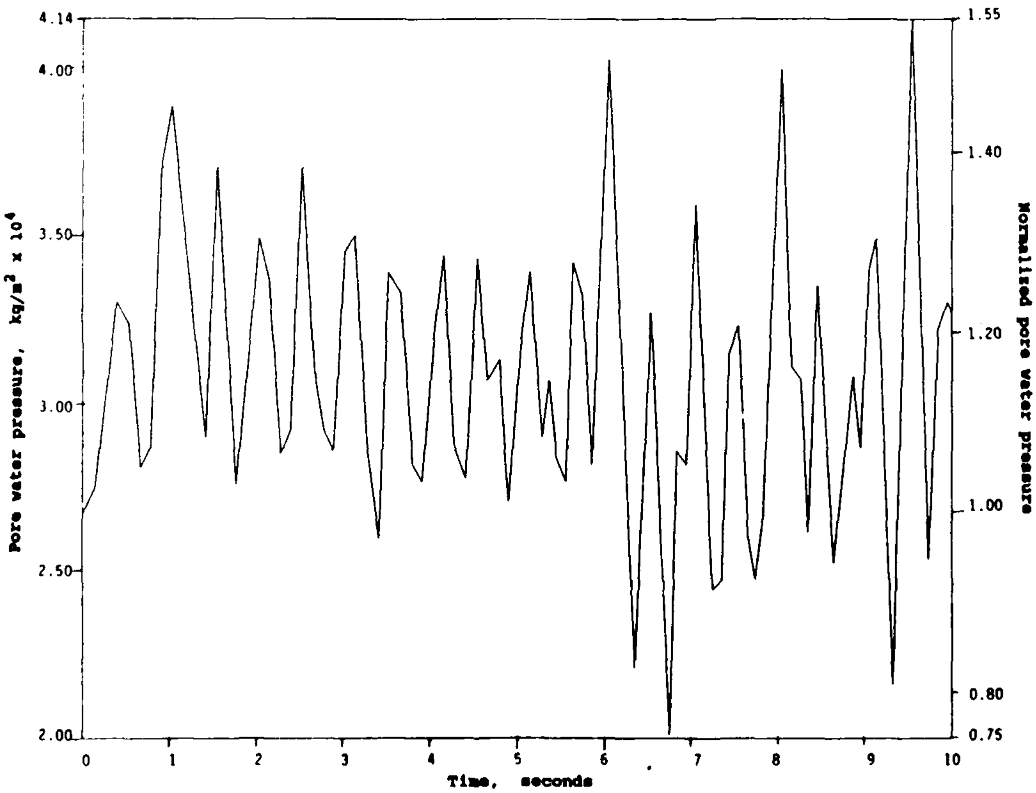


(b) Soil and stone column foundation: FOUNDATION-SC.

Figure 16. Pore water pressure in foundation element 8.



(a) Homogeneous soil foundation: FOUNDATION-H.



(b) Soil and stone column foundation: FOUNDATION-SC.
NOTE: This element is in the stone column.

Figure 17. Pore water pressure in foundation element 10.

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31 Fuel flexibility (liquid fuels, coal utilization, energy from solid waste)
32 Alternate energy source (geothermal power, photovoltaic power systems, solar systems, wind systems, energy storage systems)
33 Site data and systems integration (energy resource data, energy consumption data, integrating energy systems)

34 **ENVIRONMENTAL PROTECTION**
35 Hazardous waste minimization
36 Restoration of installations (hazardous waste)
37 Waste water management and sanitary engineering
38 Oil pollution removal and recovery
39 Air pollution

44 **OCEAN ENGINEERING**
45 Seafloor soils and foundations
46 Seafloor construction systems and operations (including diver and manipulator tools)
47 Undersea structures and materials
48 Anchors and moorings
49 Undersea power systems, electromechanical cables, and connectors
50 Pressure vessel facilities
51 Physical environment (including site surveying)
52 Ocean-based concrete structures
54 Undersea cable dynamics

TYPES OF DOCUMENTS

85 Techdata Sheets
86 Technical Reports and Technical Notes
83 Table of Contents & Index to TDS
82 NCEL Guides & Abstracts
91 Physical Security
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R. N. Storer

R. N. STORER, Ph.D, P.E.
Technical Director

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3. The report acknowledges related work accomplished by others.	( )	( )	( )	( )	( )
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